LITTLE BOW RIVER BASIN
STORAGE SITE INVESTIGATION
FEASIBILITY DESIGN REPORT

June, 1985

Prepared For:

Alberta Environment
Planning Division

Prepared By:
W-E-R Engineering Ltd.
in association with
Thurber Consultants Ltd.
and
Nanuk Engineering and Development Ltd.

July 10, 1985

Alberta Environment
Planning Division
200 - 5 Avenue South
Lethbridge, Alberta
T1J 4C7

Attention: Mr. Sandy Larsen, P.Eng.

Re: Little Bow River Basin
Storage Site Investigation
Feasibility Design Report

Dear Mr. Larsen:

W-E-R Engineering Ltd. is pleased to submit herewith fifty (50) copies of its final report "Little Bow River Basin Storage Site Investigation - Feasibility Design Report", June, 1985.

The investigation was conducted in association with Thurber Consultants Ltd. who were responsible for the geotechnical assessment, and Nanuk Engineering and Development Ltd. who carried out the water supply and hydrology studies.

The report outlines the results of feasibility design studies undertaken for three reservoir alternatives, Little Bow Site 1, Mosquito Creek Site 4 and Squaw Coulee Site 2.

Our firm appreciates the opportunity to be involved in the development of storage in this water-short area of Southern Alberta. It has been a most challenging assignment.

We would be pleased to meet with you at your convenience to discuss the results of this investigation.

Yours very truly,

W-E-R ENGINEERING LTD.

J.M. Wise, P.Eng.
Vice-President

JMW:pc

PROFESSIONAL ENGINEER ALBERTA J.M. WISE

THE ASSOCIATION OF PROFESSIONAL ENGINEERS GEOLOGISTS and GEOPHYSICISTS OF ALBERTA
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ACKNOWLEDGEMENTS

W-E-R Engineering Ltd. gratefully acknowledges the following officials and agencies for their assistance and co-operation:

- Alberta Environment, Planning Division
  - Mr. S. Larsen, P.Eng., the Study Director, and
  - Mr. L. K. Szojka, P.Eng., Oldman River Basin Planner.

- PFRA, Engineering Services
  - Mr. E. Caligiuri, P.Eng., Manager Alberta Regional Division, who provided PFRA reports and other technical information.

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W-E-R also wishes to acknowledge the involvement of their design engineer, Mr. C. H. Reynolds.

SYNOPSIS

Conceptual design studies conducted by W-E-R Engineering Ltd. on eight storage alternatives in the upper Little Bow River Basin recommended that three sites - Little Bow Site 1, Mosquito Creek Site 4 and Squaw Coulee Site 2 be retained for more detailed investigation.

This report describes the engineering studies carried out by W-E-R, which comfirm the technical feasibility of the three selected reservoir sites, and provides hydrology and water supply, geotechnical, feasibility design and reservoir damage investigations for the storages under consideration.

The total estimated costs for the three storage alternatives are as follows:

	Project Cost	Cost per Ac.Ft. of Basin Yield
Little Bow Site 1	$ 7,750,000	$215
Mosquito Creek Site 4	$10,611,000	$283
Squaw Coulee Site 2	$ 7,444,000	$219

The report recommends that the Little Bow and Squaw Coulee sites be considered for implementation and that Mosquito Creek Site 4 be deleted from further study.

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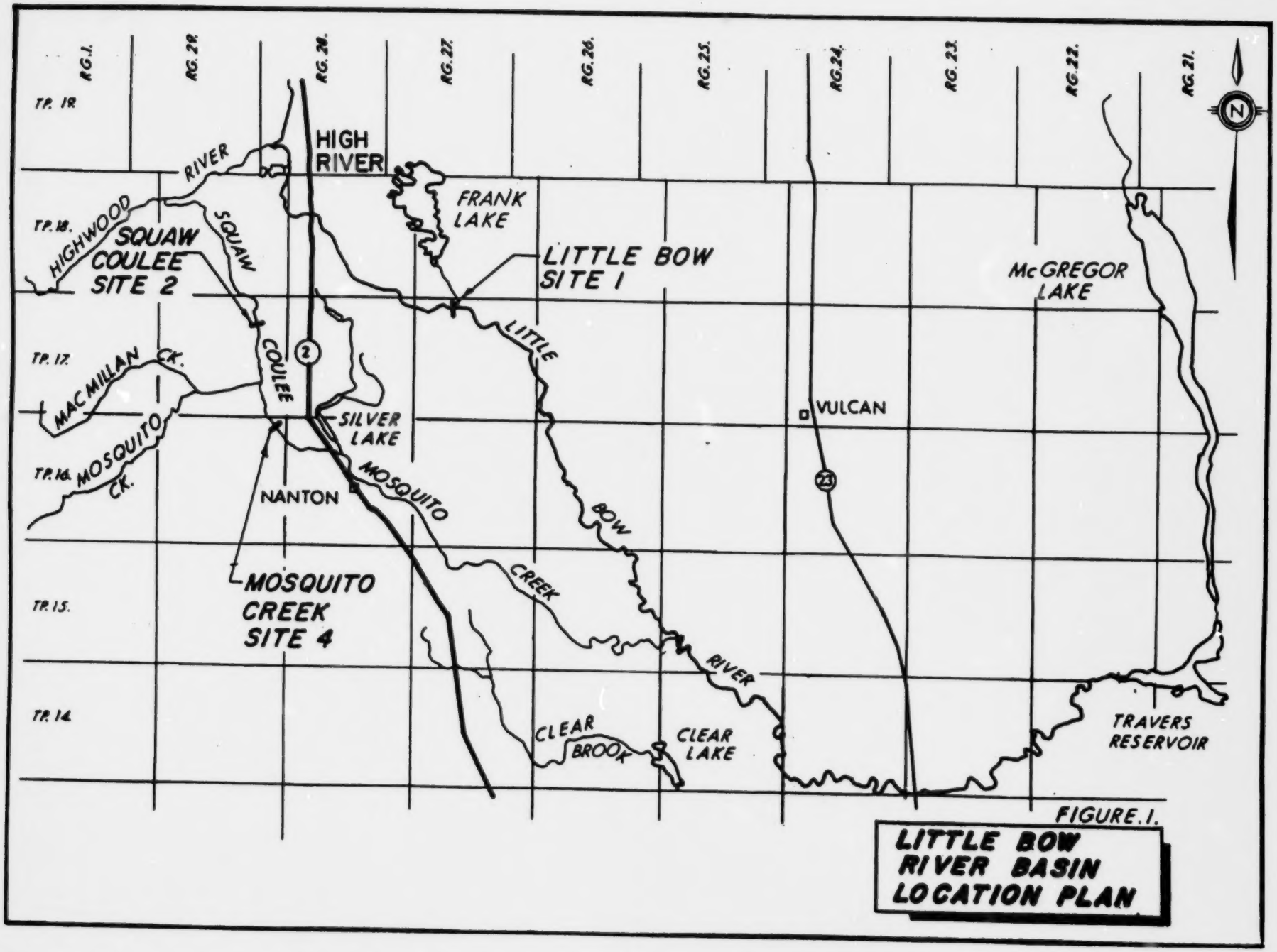

FIGURE. 1.

LITTLE BOW RIVER BASIN LOCATION PLAN

1.0 INTRODUCTION

The demand for irrigation water in the Little Bow River Basin has shown a marked increase in recent years and there are indications that this trend will probably continue. This increased use has also coincided with a period of below normal stream flows. As a result, water shortages have occurred during peak demand periods which are steadily becoming more prevalent and more severe.

Accordingly, to formulate a comprehensive water management and licencing policy for the basin, Alberta Environment instituted a basin planning study in 1982. To date a wide range of component studies dealing with various aspects of the overall problem have been completed. These background studies strongly recommended that the feasibility of constructing storage facilities in the Little Bow River Basin be investigated. W-E-R Engineering Ltd. was engaged in February, 1985 to conduct the current storage reservoir assessment in two stages.

The initial stage was a Conceptual Design Study[1] by W-E-R Engineering Ltd. which examined and compared a total of eight reservoir alternatives in the upper reaches of the Little Bow system, and recommended that three of the sites be retained for more detailed investigation. These sites, as shown in Figure 1, are Little Bow Site 1, Mosquito Creek Site 4 and Squaw Coulee Site 2.

Feasibility design investigations conducted by W-E-R Engineering for the three reservoir sites recommended for further study are now complete. These investigations present the second phase of this assignment.

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1. "Little Bow River Basin Storage Site Investigation Conceptual Design Report", May 1985

In accordance with the terms of reference established by Alberta Environment, this feasibility investigation has been limited to storage and structure sizing, for the three selected sites, based on the current 30/10 operational guideline of Highwood River flows at the Little Bow and Squaw Coulee diversions (i.e. to the maximum physical capacities of the existing diversion system - 100 cfs/2.83 m^3/s in the Little Bow Canal and 60 cfs/1.70 m^3/s at the Squaw Coulee diversion).

In following sections of this report the water supply studies, reservoir sizing, geotechnical investigations, feasibility design studies, assessments of reservoir costs and costs of implementation are summarized and the results presented. The results are also displayed in the form of structure layouts, pertinent data and cost summaries at the back of the main report.

Details of each of the study components are included in the following appendices:

- Appendix A - Water Supply and Hydrology
- Appendix B - Geotechnical
- Appendix C - Design and Costing
- Appendix D - Reservoir Costs

On the basis of the feasibility design investigations, recommendations for implementation are presented.

2.0 HYDROLOGIC ASSESSMENT

A feasibility level water supply and flood flow assessment of Little Bow Site 1, Mosquito Creek Site 4 and Squaw Coulee Site 2 was conducted as a part of the current study. The results were used in support of reservoir and spillway sizing for each of the storage sites. Details of this hydrologic assessment are provided in Appendix A.

2.1 Water Supply

The water supply capability of each of the three potential reservoir sites was evaluated in terms of the total water supply that could be made available primarily for irrigation uses, over a range of storage capacities for each reservoir.

a) Base Data

Water available from the Highwood River for the Little Bow River system was based on the assumption that a maximum of 40% of Highwood River flows could be diverted at the Little Bow and Squaw Coulee headworks up to the existing system capacities (100 cfs/2.83 m^3/s at High River, 60 cfs/1.70 m^3/s at Squaw Coulee). The operating season was assumed to be April 15 to October 15.

Natural runoffs to the Mosquito Creek Site 4 were estimated based on a correlation with gauged flows on Willow Creek compared with gauge records 1913 to 1919 on Mosquito Creek. Natural runoffs to the Little Bow Site 1 were derived from the Little Bow River gauge at Carmangay adjusted for inflows between the gauge and the reservoir site.

Reservoir unit evaporation losses were assumed to be equivalent to published evaporation rates published by PFRA for Calgary.

Irrigation demands were distributed over a five-month irrigation season in accordance with measured irrigation usage in 1984, as follows:

May	8%
June	30%
July	38%
August	19%
September	5%

Municipal and instream requirements were assumed to represent a constant year-around demand of 15 cfs (0.42 m^3/s)

b) Methodology

The period 1923 to 1941 was selected for the water supply study, since it included the most critical drought period on record. For this period of record, water balance studies were conducted for each reservoir, which considered, on an annual basis, inflows (diversions plus normal inflows) evaporation losses, and available drafts.

Based on these studies, the maximum practical size from a water supply standpoint was determined for each reservoir. However, at all three sites topographic limitations restrict the actual reservoir capacities to considerably less than these hydrologic maximums.

The studies also considered the hydrologic optimum capacities for each reservoir in terms of total basin supply. These optimum capacities occur at inflection points on the storage yield curves, (Figures A1, A2 and A3 in Appendix A), where increased storage capacities fail to produce comparable increased yields. This point occurs for all three storage sites, at about 14,000 acre-feet (17.27 x $10^6 m^3$)

of storage which will produce 34,000 acre-feet (41.94 x $10^6 m^3$) of yield.

If the Little Bow Diversion capacity is increased, the hydrologic (water supply) optimum will increase significantly. Similarly, an increase in the Squaw Coulee Diversion capacity would significantly increase the optimum storage capacity of Mosquito Site 4 or Squaw Coulee Site 2.

2.2 Flood Flows

Flood frequency analyses were carried out for use in the design of spillways and construction diversion works for each site. The probable maximum floods (PMF) for each site were determined using Saskatchewan-Nelson Basin Board data. The 1 in 100 year and 1 in 25 year floods for Mosquito Creek site, and the sites on Squaw Coulee and the Little Bow were determined using frequency analysis from different streams and independent methods of transposition. A complete description of the methodology used can be found in Appendix A.

A summary of the estimates obtained from this flood hydrology analysis is provided in Table 1.

TABLE 1
ESTIMATES OF FLOOD FLOWS

Return Period (Years)	Little Bow Site 1	Mosquito Creek Site 4	Squaw Coulee
1:25 (Peak Inst.)	700 cfs (19.82 m^3/s)	5,000 cfs (141.6 m^3/s)	600 cfs (16.99 m^3/s)
1:100 (Peak Inst.)	1,500 cfs (42.48 m^3/s)	9,500 cfs (269.0 m^3/s)	1,200 cfs (33.98 m^3/s)
PMF (Max. Daily)	7,200 cfs (203.9 m^3/s)	15,750 cfs (446.0 m^3/s)	5,600 cfs (158.6 m^3/s)

An approximate flood routing, based on the figures in Table 1, was carried out to determine outflows for structure design.

3.0 GEOLOGICAL/GEOTECHNICAL ASSESSMENT

A preliminary geotechnical assessment of the three proposed storage sites - Little Bow Site 1, Mosquito Creek Site 4 and Squaw Coulee Site 2, was carried out as a part of this investigation.

Details of the geotechnical evaluation are presented in Appendix B.

3.1 Field Work

Comprehensive drilling programs were carried out at Little Bow River Site 1 and Mosquito Creek Site 4 by PFRA. Detailed information on subsurface conditions at each of these two sites is therefore available for the geotechnical evaluation. Three test holes were recently drilled in the vicinity of Squaw Coulee Site 2 in order to permit a preliminary assessment of subsurface conditions at that site.

3.2 Site Geology

Geological conditions at all three sites are similar. On the uplands, silty clay till overlies bedrock at the sites to depths of 20 to 30 feet (6 to 9 m). The valleys are cut down into shale and sandstone bedrock which is covered on the valley walls by a thin veneer of colluvium (slopewash). Alluvial silty clays, silts, sands and gravels are present on the valley floor at all three sites. These materials are highly variable in their distribution and engineering properties. At Little Bow Site 1 and Squaw Coulee Site 2, the thickness of these alluvial sediments range up to 20 feet (6 m). At Mosquito Creek Site 4, the sediements range up to about 60 feet (18 m) in thickness. The alluvium is underlain by shale and sandstone bedrock at all three sites.

3.3 Reservoir Shoreline Stability

No significant concerns with respect to reservoir shoreline stability were identified for any of the three reservoirs. Deep seated landslides are possible in all cases, however the frequency of occurrence of such landslides is expected to be very low. Shallow instability can be expected along the reservoir shorelines due to wave erosion. The beaches are expected to be self-stabilizing over 20 to 30 years.

3.4 Reservoir Watertightness

Lateral seepage from the reservoirs is not a major concern at any of the sites. Lateral seepage losses are expected to be greatest from the Squaw Coulee reservoir, because the full supply level of this reservoir is about 100 feet (30 m) above the regional groundwater table. A buried preglacial river valley intersects the Mosquito Creek Site 4 reservoir. It is speculated that seepage from the reservoir into this valley will not be significant; however, a detailed hydrogeological investigation should be undertaken to confirm this, before a final decision is made to construct a dam at this site.

3.5 Embankment Cross-Sections

At all three damsites a zoned earth fill should be used. it is recommended that the downstream slopes be constructed at an average 3 horizontal to 1 vertical, while the upstream slopes should be constructed at an average 4 horizontal to 1 vertical. It is anticipated that the slopes can be bermed in order to reduce the quantity of fill required for the embankments. A decision in this regard will depend on the results of material balance studies, laboratory test results and stability analyses which should be undertaken during detailed design.

3.6 Seepage Below Embankments

Measures to control seepage below the dam embankments are essentially the same at Little Bow Site 1 and Squaw Coulee Site 2. At both sites, a compacted clay cutoff excavated through 15 to 20 feet (4.5 to 6 m) of pervious alluvium into bedrock is recommended. A conventional downstream drainage blanket and toe drain are also recommended at these two sites. More extensive seepage control measures are required at Mosquito Creek Site 4, because of the significant depth of alluvial sediments (60 feet) at this site. It is recommended that a sheet pile or slurry trench cutoff be installed to a depth of 40 feet (12 m) at this site to cut off the more pervious sediments. A drainage blanket and relief wells at the toe of the embankment are also recommended.

3.7 Abutment Seepage

Seepage around the abutments, through fractures and joints in the bedrock, is a concern at all three damsites. Special preparation of the abutments, as outlined in the report, will be required. It may be necessary to grout the rock at the abutments; however, further field investigations should be undertaken before a decision is made in this regard.

3.8 Embankment Cracking

Cracking of the embankment at Mosquito Creek Site 4, as a result of consolidation of the underlying alluvium, is a concern. Measures to mitigate the possible effects of cracking at this site include the use of an upstream zone of well graded gravelly silt, a chimney drain in the downstream portion of the embankment and compaction of the impervious

clay fill at a water content which is wet of optimum. Such measures are not considered necessary at Little Bow Site 1 or at Squaw Coulee Site 2.

3.9 Spillways

A single spillway, excavated around the dam abutment, is being proposed at all three of the sites. The spillway at Little Bow Site 1 will be excavated through about 15 feet (4.5 m) of clay till and 15 feet (4.5 m) into the underlying shale and sandstone bedrock. The excavation for the spillway at Mosquito Creek Site 4 will extend through about 25 feet (7.5 m) of clay till and about 5 feet (1.5 m) into the underlying bedrock. The spillway excavation at Squaw Coulee Site 2 will be made through several feet of colluvium and about 25 feet (7.5 m) into the underlying bedrock. No significant construction difficulties with any of these excavations are anticipated. Blasting of cemented sandstone layers will be required if such material is encountered within the bedrock. Permanent slopes in clay till should be cut at 3 horizontal to 1 vertical. Permanent slopes in the bedrock should be cut at 2 horizontal to 1 vertical, primarily to minimize surface erosion of the shale. If steeper slopes are desired in the bedrock, surface treatment, such as shotcrete, should be considered.

3.10 Low Level Conduits

The low level conduits at all three sites should be founded either on bedrock or on compacted silty clay fill which extends below the conduit down to the bedrock. The conduit should therefore be located where the depth to bedrock is a minimum at all three sites. In the case of Mosquito Creek Site 4, the preferred location is adjacent to the north abutment.

3.11 Concrete

Sources for concrete aggregate have not been confirmed at any of the sites. It is anticipated that suitable aggregate (which will have to be crushed, screened and washed) can be located in the alluvial sediments upstream from Little Bow Site 1 and Mosquito Creek Site 4. No source for concrete in the immediate vicinity of Squaw Coulee Site 2 has been identified. It is recommended that Type 50 sulphate resistant cement be used for all concrete in contact with soil at all three sites.

3.12 Additional Geotechnical Investigations

Prior to final design, a comprehensive drilling program will be required at Squaw Coulee Site 2. Such work is not necessary at Little Bow Site 1 or Mosquito Creek Site 4 since this work was carried out previously by PFRA. Supplementary geotechnical investigations and analyses would, however, be required at all three sites, as described in the report.

4.0 DESIGN STUDIES

4.1 General

Pre-design studies were carried out for each of the three damsites, which covered structure layouts, hydraulic and structural design and costing of the main dam and appurtenant works. Details of the design studies are presented in Appendix C.

These studies utilized the current hydrology and water supply, and geotechnical assessments in support of designs for the main dam, spillway and conduit structures.

Topographic mapping available from the 1968 PFRA report was used for the damsites and reservoirs at Little Bow Site 1 and Mosquito Creek Site 4. New photogrammetric mapping was generated for Squaw Coulee Site 2.

4.2 Reservoir Sizing

Reservoir FSL's were set at the topographic or physical maximum for all three storage sites. This was a limitation established in the conceptual design study, with the reservoir sizes corresponding approximately to the water supply optimum as discussed in the hydrology section.

4.3 Project Requirements

Aside from the establishment of reservoir size for each damsite, basic project requirements for design included: a) controlling floods which were set at 0.5 PMF for the safety-of-dam and 1:100 for project design purposes, and b) downstream irrigation requirements which were established as 300 cfs (8.49 m^3/s) for the three sites.

4.4 General Layout

The general arrangements of the dam and appurtenant structures is shown in Figures 2, 3 and 4 with pertinent data provided in Tables 2, 3 and 4. Dam centreline locations for the Little Bow and Mosquito Creek sites are the same as those used in the earlier PFRA study. All structure layouts have been selected to take advantage of the topographic and geological conditions at each site. Embankment designs are based on the geotechnical investigations presented in this report.

4.5 Spillways

Reinforced concrete, gated chute type spillways with hydraulic jump stilling basins are proposed for handling floods. The structure would be equipped with automatically operated radial gates.

Spillway and gate size combinations have been optimized to provide the most cost effective arrangement. Chute structures are flared into the stilling basin to ensure that flow energy is adequately dissipated.

Spillway structural components have been selected on the basis of existing structures of a similar type. Conventional cutoffs, key walls, joints and drainage systems have been incorporated into the design to accommodate movements and control foundation seepage.

4.6 Irrigation Conduits

Irrigation conduits are basically low level outlet structures designed to handle the downstream irrigation demand.

Precast concrete pipelines with reinforced concrete centre gatewells and appropriate inlet and outlet structures are proposed. Gatewells will be two-celled structures equipped with operating and emergency gates for control of irrigation releases. The conduits will also be used for construction diversion purposes prior to conversion to permanent outlets.

4.7 Construction Cost Estimates

Cost summaries, including costs of the main works, are provided in Tables 2, 3 and 4. Cost estimates for the main dam and structures include allowances for contingencies and engineering. Unit prices used in preparing cost estimates are based on recent bid prices on similar projects in Southern Alberta.

4.8 Project Implementation

It is estimated that final designs and geotechnical studies and the preparation of tender documents can be completed in about four months. Construction of any of the three damsites under consideration will require two seasons. Ideally, designs could be carried out over winter with tenders called the following spring. Winter construction of structural components would also be feasible.

5.0 RESERVOIR COSTS

An estimate of the costs of land acquisition and reservoir damages has been prepared for each of the three reservoir sites, as summarized in following sections of this report. Details of the reservoir costs are included in Appendix D.

5.1 Basis of Costing

Reservoir land costs are based on the total land control requirements taken to the crest of the dam and include an allowance for severance and acquisition.

Reservoir damages include the cost of roads and bridges, power lines, gas pipelines, AGT lines and relocation or dyking of farm buildings.

Total reservoir costs for each site are provided in Tables 2, 3 and 4.

5.2 Little Bow Site 1

a) **Land Acquisition** - A total of 892 acres (361 ha) of land to the top-of-dam elevation are affected, 250 acres (101.2 ha) of which are cultivated.

b) **Roads** - One existing road and bridge at the east boundary Section 3-18-24 W4 would have to be raised by about five feet (1.5 m).

c) **Utilities** - No power lines, pipelines or telephone cables have been identified in the reservoir area other than for farm service to farmsteads which would be relocated.

d) **Buildings** - Three sets of farm buildings are located within the reservoir area.

5.3 Mosquito Creek Site 4

a) **Land Acquisition** - There are 1,530 acres (619.2 ha) of land located within the reservoir of which 580 acres (234.7 ha) are under cultivation.

b) **Roads** - An existing road and bridge located at the north boundary of Section 35-16-29-W4 would be relocated over the top of the dam. This would involve constructing approximately three miles (4.8 km) of new road, and amending the spillway design to accommodate road traffic over the operating deck.

A second road crossing at the north boundary of Section 11-17-29-W4 would have to be raised by 15 feet (4.6 m) above the present elevation. A road along the west boundary of Section 1-17-29-W4 would also have to be raised and a drainage pumpstation in the low area on the east side of the road, would have to be constructed.

c) **Utilities** - A Trans-Alta 500 KvA line crosses the reservoir at the north boundary of Section 2-17-29-4. Trans-Alta estimates that special long span support towers might be feasible provided that a minimum 35 feet (10.7 m) of ground clearance can be maintained. The alternative would be an expensive three-mile (4.8 km) long relocation of the power.

A small powerline crosses the reservoir at the north boundary of 11-17-29-W4. To re-route this line around the north end of the reservoir would require the construction of about two miles (3.7 km) of line at an estimated cost of $20,000.

A 50 mm diameter gas pipeline crosses the reservoir in Section 35 and 36-16-29-W4. To re-route this line over the dam would require 1.1 miles (1.8 km) of relocation.

Relocation of a small farm service pipeline in Sections 9 and 10-17-29-W4 would also be required.

A buried telephone cable is located at the north boundary of Section 11-17-29-W4. Relocating this line would involve three miles (4.8 km) of new buried cable installation.

d) **Farm Buildings** - Three sets of farm buildings are located within the reservoir "take line". Two of these, in the SE 14-17-29-W4 and NW 36-16-29-W4, could be left in place with the construction of protective dykes.

A farmstead located in NE 2-17-29-W4 is too low to be dyked off from the reservoir and would therefore be relocated.

5.4 Squaw Coulee Site 2

a) **Land Acquisition** - A total area of 730 acres (295.4 ha) is affected by the Squaw Coulee Site 2 reservoir.

b) **Roads** - Secondary Highway 540, which crosses the reservoir at the north boundary of Section 34-17-29-W4 would have to be raised by 33 feet (10.1 m).

5.5 Archaeological Sites at Squaw Coulee

Informal discussions were held with archaeologists at the University of Calgary and with the Archaeological Survey of Alberta (Alberta Culture) to determine the location of the "Old Women's Buffalo Jump" in Squaw Coulee. The jump is considered an historic site and the objective of the inquiries was to determine whether it would be inundated by the proposed reservoir at Squaw Coulee Site 2.

The site is located on the boundary between LSD 12 and LSD 13-35-17-29-W4M, one quarter of a mile south of Highway 540 on the south rim of the coulee. The jump itself is above the top-of-dam elevation but an addition site associated with the jump is located at the bottom of the coulee. This additional site would probably have to be mitigated should the Squaw Coulee 2 site proceed.

It was also pointed out that the coulee, like all other glacial meltwater channels, may have many other archaeological sites.

The costs associated with mitigating this site and any others have not been included in the reservoir costs, and would have to be investigated further prior to implementation.

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

1. It is technically feasible to construct storages of the size recommended at Little Bow Site 1, Mosquito Creek Site 4 and Squaw Coulee Site 2.

2. The total estimated costs for the three storages are as follows:

	Project Cost	Cost/Acre-Foot of Yield
Little Bow Site 1	$ 7,750,000	$215
Mosquito Creek Site 4	$10,611,000	$283
Squaw Coulee Site 2	$ 7,444,000	$219

3. The total project costs have risen considerably for the Mosquito Creek and Squaw Coulee sites. This was due primarily to geotechnical and reservoir damage changes at both sites in addition to new survey data in the case of Squaw Coulee.

4. Based on the cost per acre-foot of basin yield, the Little Bow and Squaw Coulee sites are substantially more economical than the Mosquito Creek site.

5. Reservoir costs at Mosquito Creek are considerably higher than the other two sites.

6. Reservoir integrity at Mosquito Creek is a concern.

7. Archaeological sites along Squaw Coulee will require further investigation prior to implementation of the site.

8. The Little Bow site probably represents the most effective use of storage considering the diversion guideline.

6.2 Recommendations

Based on the feasibility design studies conducted for the three sites and the conclusions reached, it is recommended that:

1. Further consideration be given to the implementation of the Little Bow and Squaw Coulee sites and that the Mosquito Creek site be deleted from further study.

2. More detailed geotechnical investigations be conducted at Squaw Coulee Site 2.

3. Detailed reservoir integrity studies be carried out for both the Little Bow and Squaw Coulee sites.

7.0 OBSERVATIONS

The conceptual design studies did not consider the feasibility of constructing more than one reservoir (for example one reservoir on the Little Bow River and one on Mosquito Creek). Subsequent studies might address this aspect and compare the technical feasibility and water supply capability of a two-reservoir concept in comparison with a single large reservoir below the confluence.

Finally, it should be noted that the study results as presented in this report are based on the current 10/30 diversion guideline of flows from the Highwood River. Should these guidelines be increased in the future, this could alter the results of the current study.

TABLE 2
LITTLE BOW SITE 1

PROJECT DATA

Reservoir

Full Supply Level (FSL)	3,290 ft. (1002.8 m)
Total Storage at FSL	17,250 ac.ft. (21.28 x $10^6 m^3$)
Flooded Area at FSL	810 ac. (328 ha)
Probable Maximum Flood (inflow)	7,200 cfs (203.9 m^3/s)
Project Design Flood - 0.5 PMF (inflow)	3,600 cfs (102.0 m^3/s)

Embankment

Top of Dam Elevation	3,299 ft. (1005.5 m)
Maximum Height	59 ft. (18.0 m)
Volume of Fill	662,000 $yds.^3$ (506.2 x $10^3 m^3$)
Freeboard at FSL	9 ft. (2.7 m)

Spillway

Crest Elevation	3,285.1 ft. (1001.9 m)
Radial Gates	8.2 ft. H x 4.9 ft. W
Basin Elevation	3,235 ft. (986.0 m)
Design Discharge (1:100 est. routed outflow)	1,000 cfs (28.3 m^3/s)
Probable Maximum Discharge - 0.5 PMF (routed outflow)	2,160 cfs (61.2 m^3/s)

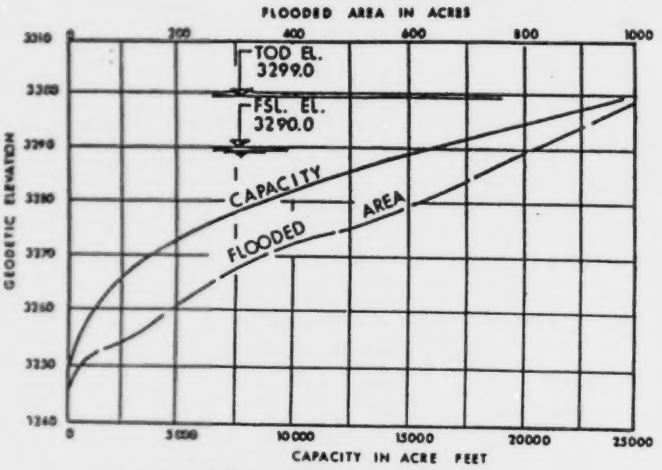
Irrigation Supply Conduit

Size of Conduit	5 ft. dia. (1524 mm)
Size of Gates	6' x 6' (1.83 m x 1.83 m)
Permanent Outlet Capacity	300 cfs (8.5 m^3/s)
Intake Elevation	3,243 ft. (988.5 m)
Reservoir Elevation to Pass Design Capacity	3,257 ft. (992.7 m)

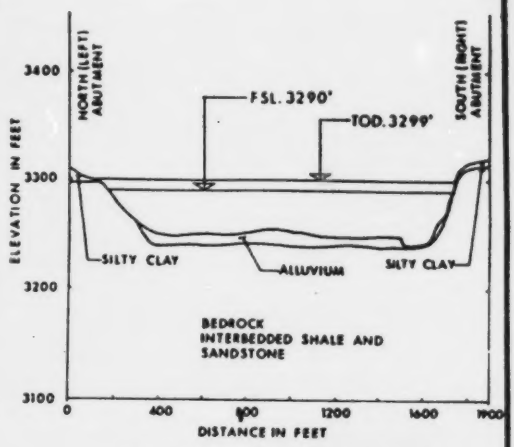
COST ESTIMATE SUMMARY

Care of Water		219,000
Embankment		2,975,000
Spillway		876,000
Irrigation Supply Works		532,000
	Subtotal	4,602,000
	Contingencies (20%)	920,000
	Subtotal	5,522,000
	Engineering (15%)	828,000
	Reservoir Costs	1,400,000
	TOTAL PROJECT COST	$7,750,000

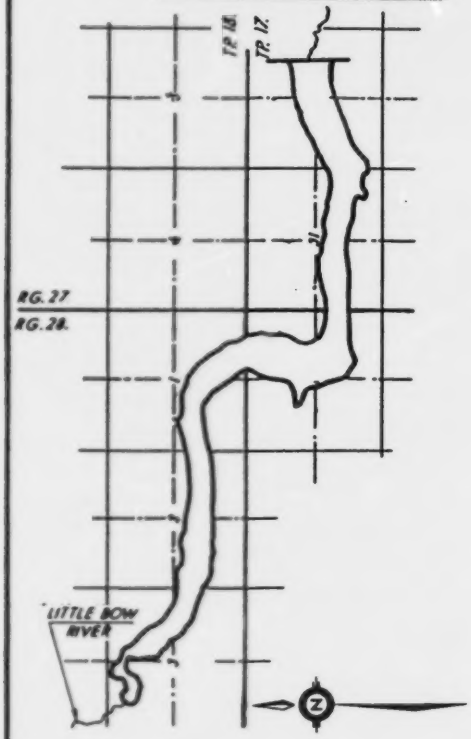
LITTLE BOW SITE 1



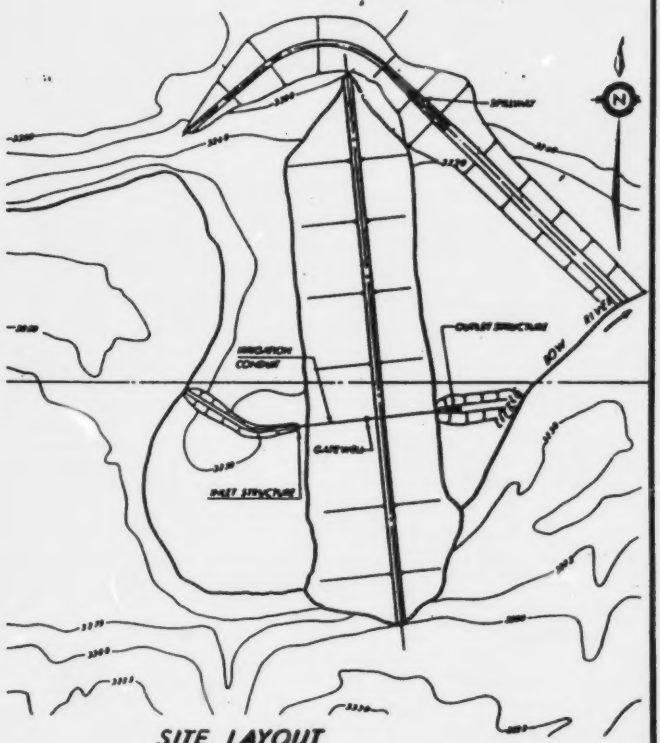
FLOODED AREA AND CAPACITY CURVES



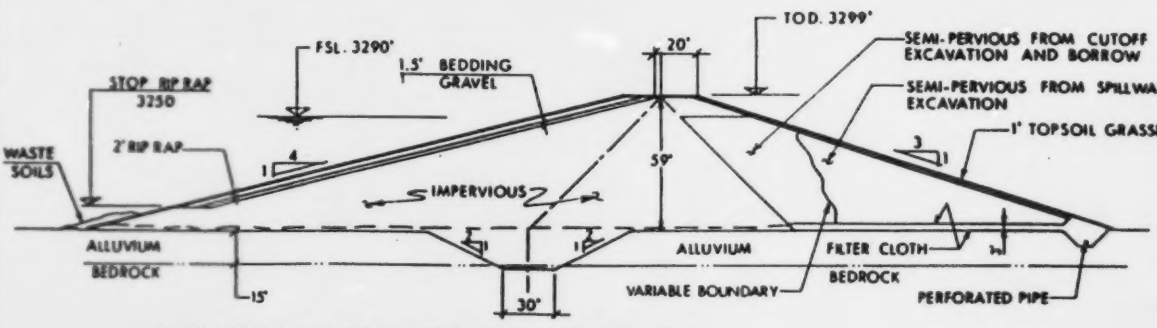
VALLEY CROSS SECTION AT DAMSITE



LOCATION PLAN



SITE LAYOUT



MAXIMUM EMBANKMENT SECTION

TABLE 3
MOSQUITO CREEK SITE 4

PROJECT DATA

Reservoir

Full Supply Level (FSL)	3,423 ft. (1043.3 m)
Total Storage at FSL	21,000 ac.ft. (25.9 x $10^6 m^3$)
Flooded Area at FSL	1,200 ac. (486 ha)
Probable Maximum Flood (inflow)	15,800 cfs (447.4 m^3/s)
Project Design Flood - 0.5 PMF (inflow)	7,900 cfs (223.7 m^3/s)

Embankment

Top of Dam Elevation	3,433 ft. (1046.4 m)
Maximum Height	65 ft. (19.8 m)
Volume of Fill	466,000 $yds.^3$ (426.1 x 10^3 m)
Freeboard at FSL	10 ft. (3.0 m)

Spillway

Crest Elevation	3,418.3 ft.
Radial Gates	8.2' W. x 4.9' H
Basin Elevation	3,366 ft. (1026.3 m)
Design Discharge (1:100 est. routed outflow)	5,000 cfs (141.6 m^3/s)
Probable Maximum Discharge - 0.5 PMF (routed outflow)	6,320 cfs (179.0 m^3/s)

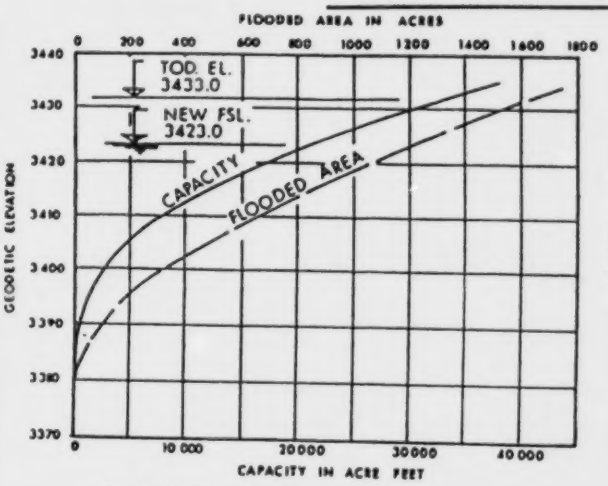
Irrigation Supply Conduit

Size of Conduit	5' dia. (1524 mm)
Size of Gates	6' x 6' (1.83 m x 1.83 m)
Permanent Outlet Capacity	300 cfs (8.5 m^3/s)
Inake Elevation	3,374 ft. (1028.4 m)
Reservoir Elevation to Pass Design Capacity	3,388 ft. (1032.7 m)

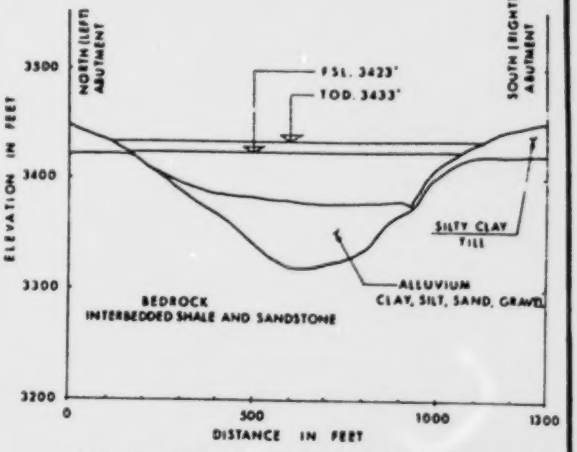
COST ESTIMATE SUMMARY

Care of Water		235,000
Embankment		2,448,000
Spillway		1,589,000
Irrigation Supply Works		662,000
	Subtotal	4,934,000
	Contingencies (20%)	987,000
	Subtotal	5,921,000
	Engineering (15%)	888,000
	Reservoir Costs	3,802,000
	TOTAL PROJECT COST	$10,611,000

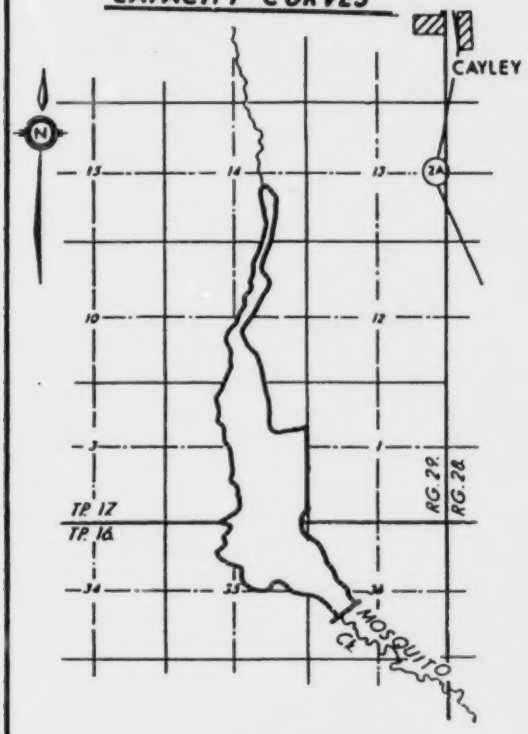
MOSQUITO CREEK SITE 4



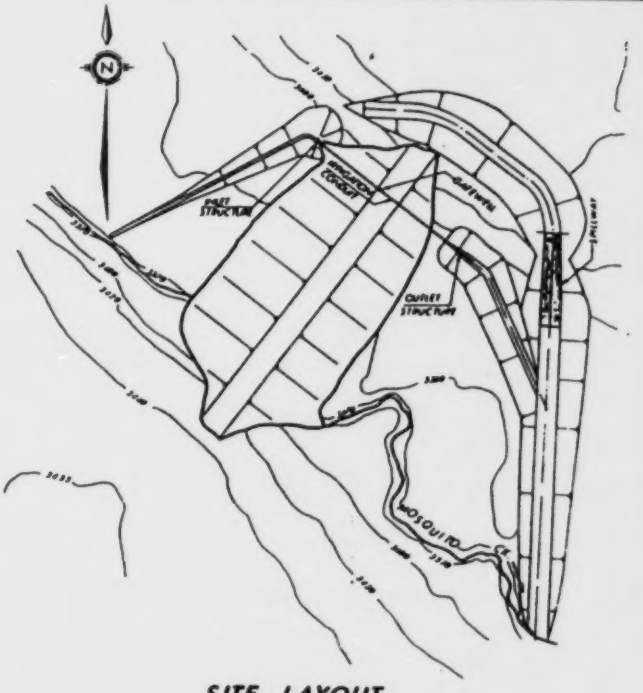
FLOODED AREA AND CAPACITY CURVES



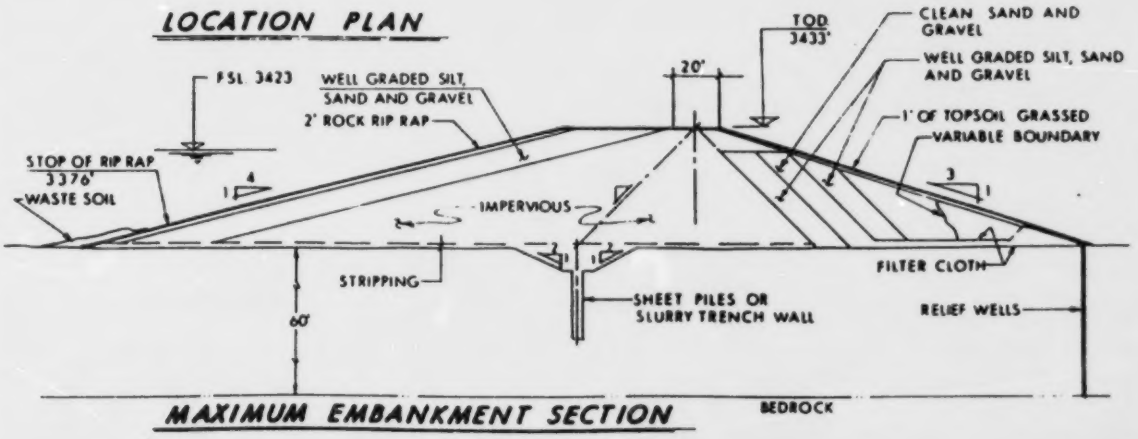
VALLEY CROSS SECTION AT DAMSITE



LOCATION PLAN



SITE LAYOUT



MAXIMUM EMBANKMENT SECTION

TABLE 4
SQUAW COULEE SITE 2

PROJECT DATA

Reservoir

Full Supply Level (FSL)	3,500 ft. (1066.8 m)
Total Storage at FSL	14,400 ac.ft. (17.76 x $10^6 m^3$)
Flooded Area at FSL	600 ac. (243 ha)
Probable Maximum Flood (inflow)	5,900 cfs (167.1 m^3/s)
Project Design Flood - 0.5 PMF (inflow)	2,950 cfs (83.53 m^3/s)

Embankment

Top of Dam Elevation	3,509 ft. (1069.5 m)
Maximum Height	69 ft. (21.0 m)
Volume of Fill	440,000 $yds.^3$ (402.3 x 10^3 m)
Freeboard at FSL	9 ft. (2.7 m)

Spillway

Crest Elevation	3,497 ft. (1065.9 m)
Radial Gates	8.2 ft. W x 4.9' H (2.5 m x 1.5 m)
Basin Elevation	3,437 ft. (1047.6 m)
Design Discharge (1:100 est. routed outflow)	900 cfs (25.5 m^3/s)
Probable Maximum Discharge - 0.5 PMF (routed outflow)	1,770 cfs (50.1 m^3/s)

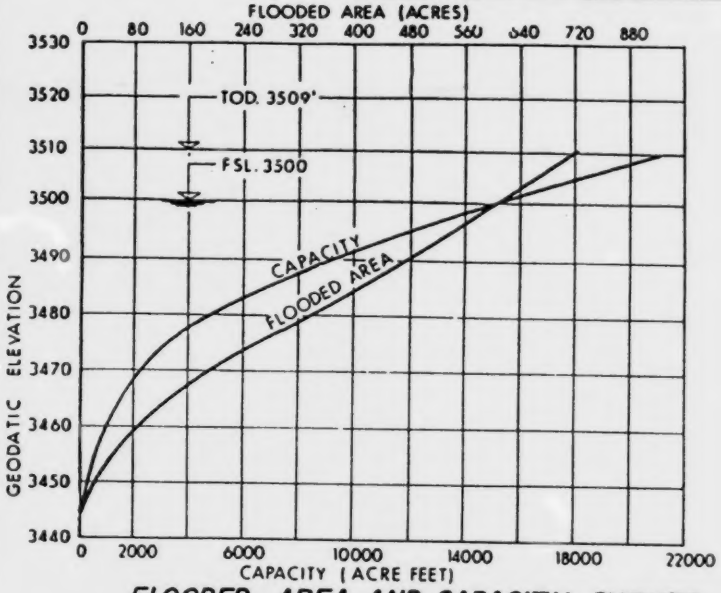
Irrigation Supply Conduit

Size of Conduit	5 ft. dia. (1524 mm)
Size of Gates	6' x 6' (1.83 m x 1.83 m)
Permanent Outlet Capacity	300 cfs (8.5 m^3/s)
Intake Elevation	3,445 ft. (1050.0 m)
Reservoir Elevation to Pass Design Capacity	3,459 ft. (1054.3 m)

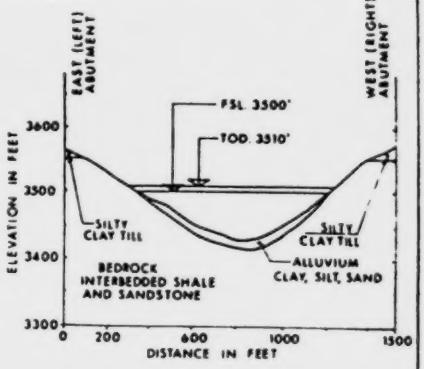
COST ESTIMATE SUMMARY

Care of Water		174,000
Embankment		1,848,000
Spillway		1,054,000
Irrigation Supply Works		575,000
	Subtotal	3,651,000
	Contingencies (20%)	730,000
	Subtotal	4,381,000
	Engineering (15%)	657,000
	Reservoir Costs	2,406,000
	TOTAL PROJECT COST	$7,444,000

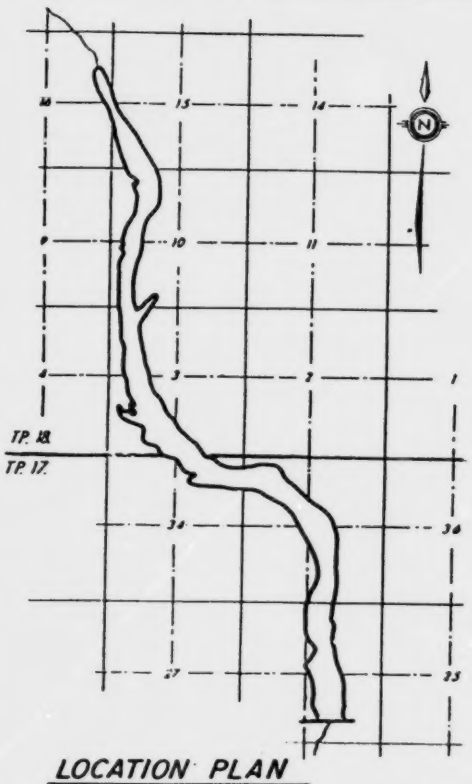
SQUAW COULEE SITE 2



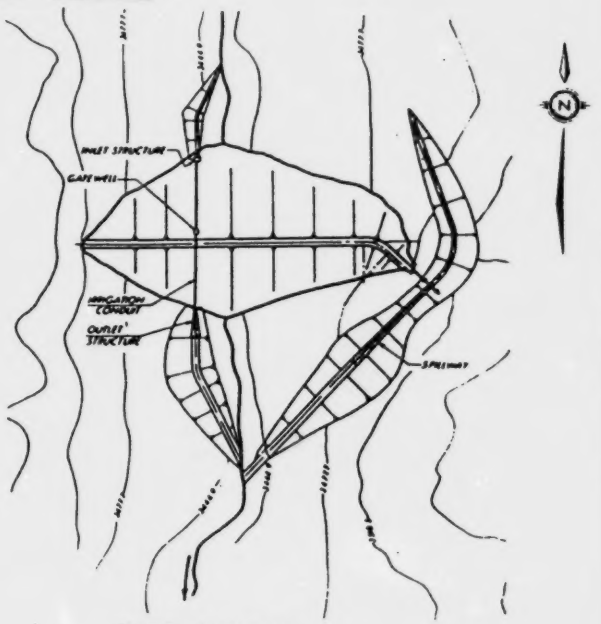
FLOODED AREA AND CAPACITY CURVES



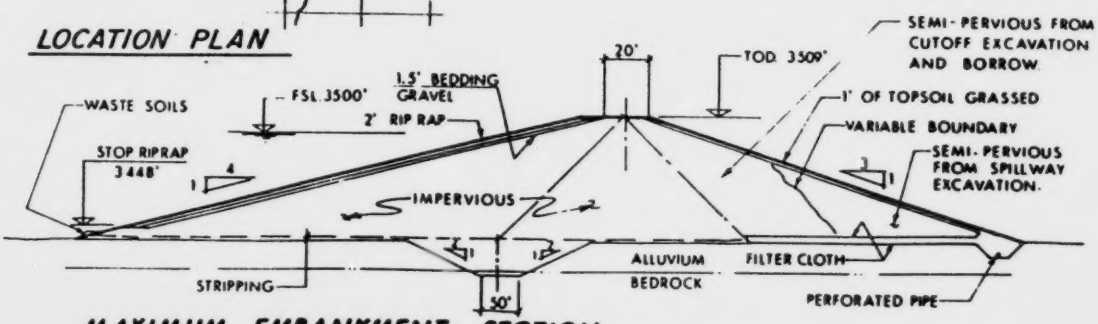
VALLEY CROSS SECTION AT DAMSITE



LOCATION PLAN



SITE LAYOUT



MAXIMUM EMBANKMENT SECTION

LIST OF APPENDICES

A - WATER SUPPLY AND HYDROLOGY
(Nanuk Engineering and Development Ltd.)

B - GEOTECHNICAL EVALUATION
(Thurber Consultants Ltd.)

C - DETAILED SUMMARY OF DESIGN STUDIES AND COST ESTIMATES

D - RESERVOIR COSTS

LITTLE BOW RIVER BASIN
STORAGE SITE INVESTIGATION
FEASIBILITY DESIGN REPORT
June, 1985

APPENDIX A

WATER SUPPLY AND HYDROLOGY STUDY

Box 817
Cochrane, Alberta T0L 0W0
Telephone (403) 932-3593

RECEIVED
MAY 30 1985

NANUK ENGINEERING & DEVELOPMENT LTD.

May 29, 1985

WER Engineering
Horizon 39 Building, Room 306
2735 39 Avenue NE
Calgary, Alberta
T1Y 4T8

Attention: Jack Wise

RE: LITTLE BOW RIVER BASIN; STORAGE INVESTIGATION STUDY
WATER SUPPLY AND FLOOD HYDROLOGY

Dear Jack;

We forward, herein, a report on the assessment of water supply and flood hydrology for three potential storage sites, Little Bow No. 1, Squaw Coulee No. 2 and Mosquito Creek No. 4.

The water supply available to the storage sites was based on available diversion flows from the Highwood River calculated by Hydrocon Engineering, 'Water Supply Assessment for the Little Bow River Basin'. Natural flows into the reservoirs were estimated and losses to reservoir evaporation were considered. Channel losses were not taken into account. The storage capacities at the three sites were evaluated based on th total basin water demand that can be satisfied by diversion, natural flows and reservoir releases. The water demand consisted, primarily, of irrigation usage.

Estimates of the probable maximum, 1 in 100 and 1 in 25 year floods were improved from the Phase I study using data from West Arrowwood and Nanton Creeks. The flood hydrology is adequate for preparing cost estimates for flood management structures but should not be used for design purposes.

Yours truly,

M.V. Thompson, P.Eng.
Chief Engineer

MVT:lc
attachment

PROFESSIONAL ENGINEER M. V. THOMPSON

PERMIT TO PRACTICE
NANUK ENGINEERING & DEVELOPMENT LTD.
Signature M.V. Thompson
Date May 30, 1985
PERMIT NUMBER: P 2506
The Association of Professional Engineers, Geologists and Geophysicists of Alberta

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	1.3 Methodology	A-3
	1.4 Results	A-4
2.0	FLOOD HYDROLOGY	A-5
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	2.2 Squaw Coulee #2 and Little Bow #1	A-6

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2	Estimates of Flood Flows

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A2	Total Basin Supply vs. Live Storage Mosquito Creek Site 4
A3	Total Basin Supply vs. Live Storage Squaw Coulee Site 2

1.0 WATER SUPPLY

1.1 Basis of Evaluation

The three potential sites for reservoir storage were evaluated for the ratios of the total basin supply to storage volume at various levels of storage capacity. Total basin supply is the total volume of water available in the basin to meet water requirements, primarily irrigation demand. The bases of these evluations were:

1. The water available for diversion from the Highwood River was in accordance with existing guidelines (10% of Highwood flows at Squaw Coulee and 30% of Highwood flows at the Little Bow Diversion) as calculated by Hydrocon Engineering in Scenario "E" except that the total diversion could be routed, up to the physical capacity, through either Squaw Coulee or the Little Bow Canal;

2. The physical capabilities of the diversions are 60 cfs at Squaw Coulee and 100 cfs in the Little Bow Canal;

3. The diversion canals were assumed to operate from April 15 to October 15 to provide water for irrigation and to the reservoir;

4. The municipal and riparian flow requirement of 15 cfs will be satisfied during the winter by diversion through the Little Bow Canal;

5. Natural flows were calculated on an annual basis and were assumed to enter the reservoir in April of each year;

6. The draft from the reservoir and diversions served a variable irrigation demand and a uniform municipal, instream use and residual flow requirement;

7. The seasonal irrigation demand was distributed in accordance with the 1984 withdrawals as follows:

May	8%
June	30%
July	38%
August	19%
September	5%

8. evaporative losses were calculated on an annual basis and were assumed to enter the reservoir in April of each year;

9. water demands were considered to be downstream of all three sites.

1.2 Natural Flows, 1916 - 1941

The natural flows in Mosquito Creek were calculated by regression on the flows of Willow Creek at Claresholm. The gauged flows of Mosquito Creek at Nanton, 1913 - 1919, from April through October were correlated with the seasonal flows in Willow Creek. The correlation coefficient was 0.966. The regression equation produced an average, seven-month flow of 15,700 ac.ft. at Nanton. The unit seasonal runoff for Mosquito Creek calculated in this manner is provided in Table 1.

The natural flows into the mainstem of the Little Bow were derived by subtracting the estimated flows of Mosquito Creek

TABLE 1

TOTAL RUNOFF
(ac.ft./sq.mi/year)

YEAR	MOSQUITO CREEK	LITTLE BOW
1916	224	0
1917	263	0
1918	24	2
1919	8	12
1920	132	85
1921	14	15
1922	39	0
1923	144	2
1924	39	3
1925	51	12
1926	93	8
1927	225	31
1928	110	85
1929	114	8
1930	51	9
1931	0	12
1932	76	4
1933	55	13
1934	29	16
1935	21	9
1936	5	31
1937	7	7
1938	81	5
1939	25	12
1940	33	10
1941	16	12
	---	---
MEAN	72	16
	===	===

at Nanton from the estimated natural flows of the Little Bow at Carmangay (South Saskatchewan River Basin Planning Program). When Mosquito Creek flows exceeded Little Bow values (7 instances in 26 years), the Mosquito Creek flows were made equal to the Little Bow flows. The resultant seasonal, unit runoffs are shown in Table 1.

1.3 Methodology

The method of evaluation used the critical period 1923 to 1938 to calculate the water balance in the reservoir. The initial part of this period was used to check that the reservoir could be filled by the fall of 1928. The latter part represents the most critical drought within the period of record.

The diversions to the Little Bow Basin were made in the most beneficial manner within the constraints of Scenario 'E' for each reservoir. If the reservoir was on Squaw Coulee or Mosquito Creek, the diversion through Squaw Coulee was made equal to the physical diversion capacity or the total flow available for diversion, whichever was the lesser.

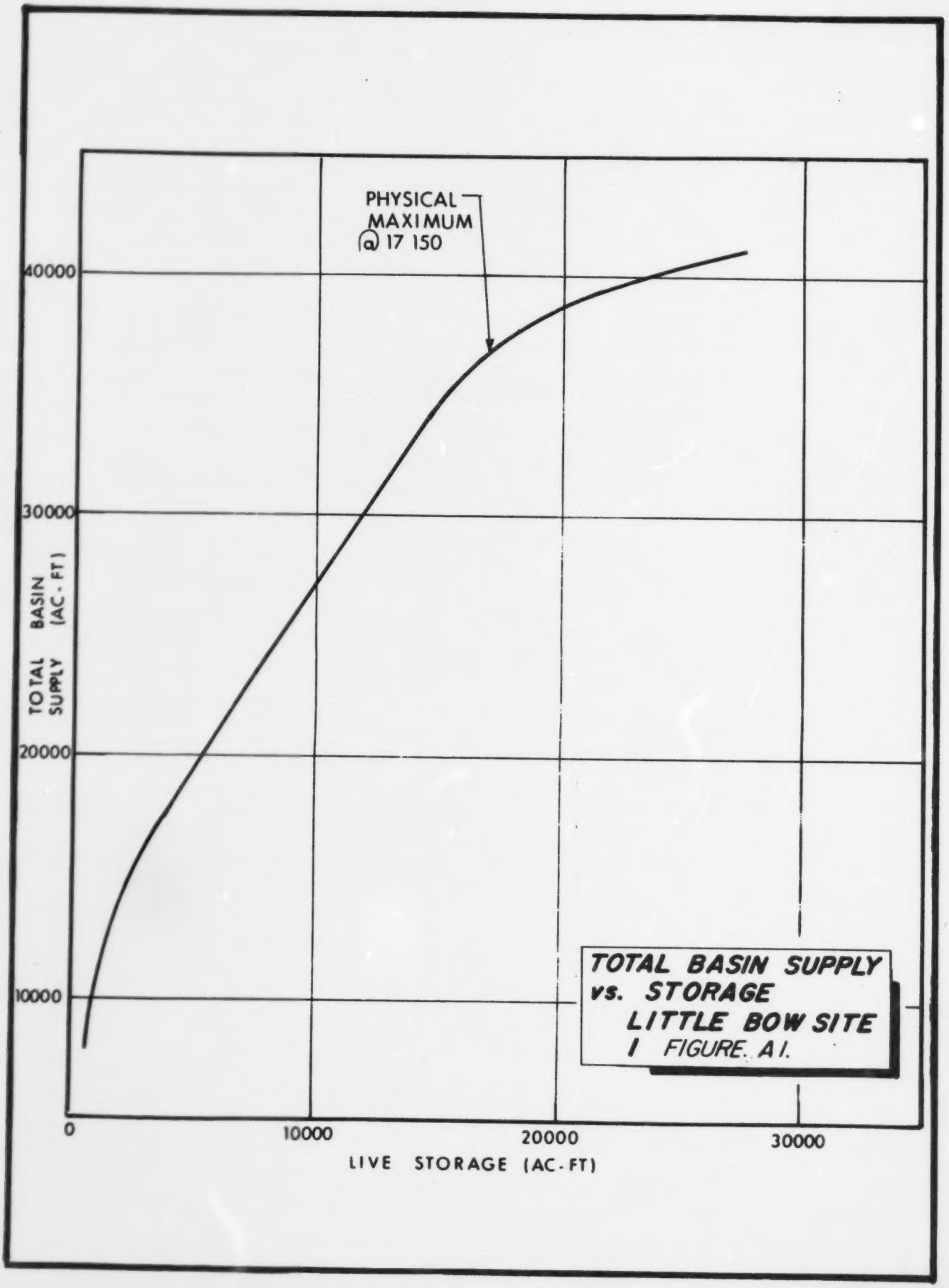
The maximum practical size of a reservoir was established by determining the maximum volume available to fill a reservoir in the period 1923 to 1928 consistent with the maximum water demand that can be satisfied in the period 1931 to 1938. This period was chosen since it contains the first and third ranked "DRY" years in terms of total diversions at the Little Bow Canal for Scenario 'E' (Hydrocon Engineering, "Water Supply Assessment for the Little Bow River Basin").

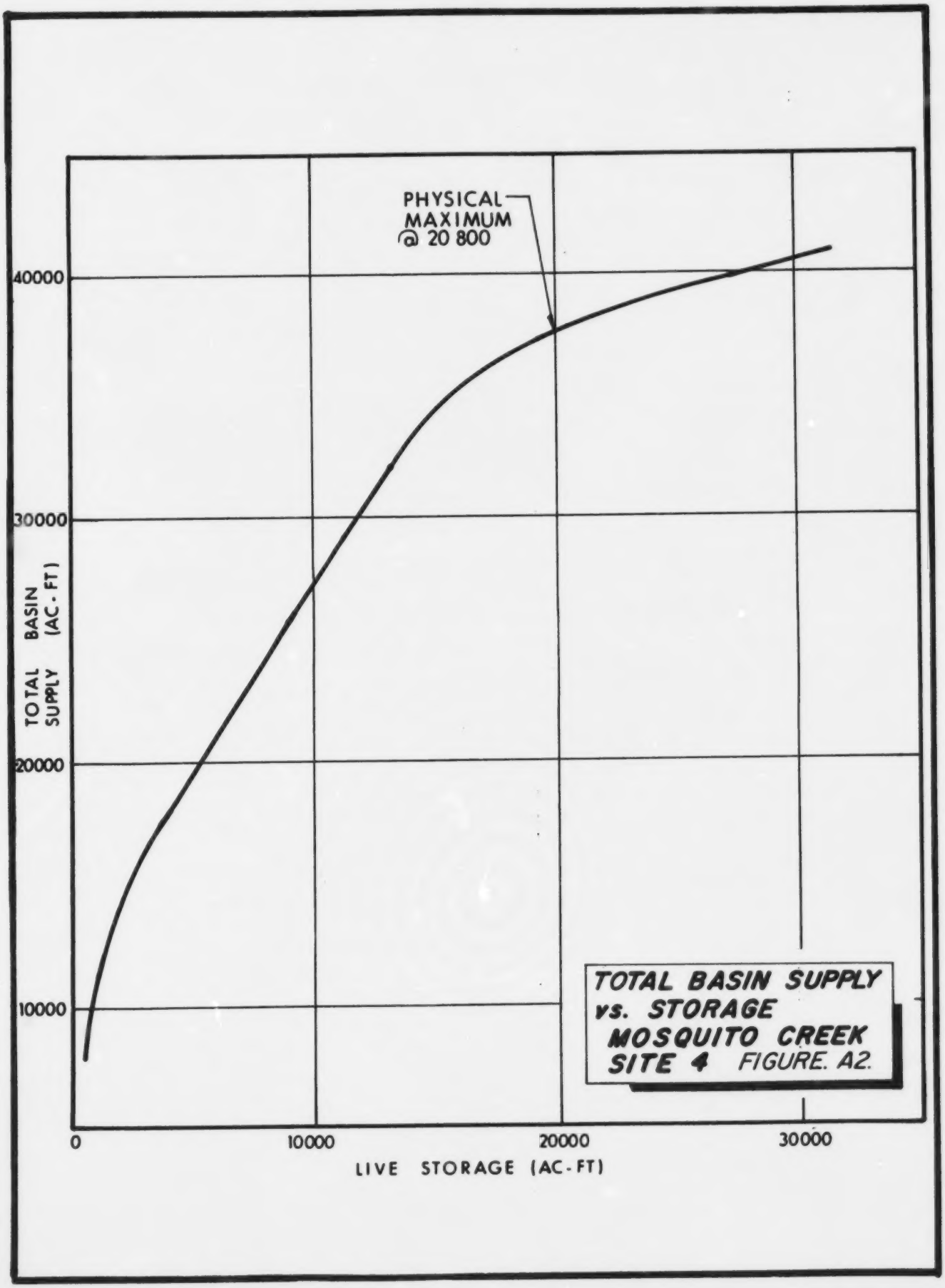
1.4 Results

The results of the evaluation are displayed on Figures A1, A2 and A3. The maximum practical reservoir sizes based on diversion flows without evaporation are:

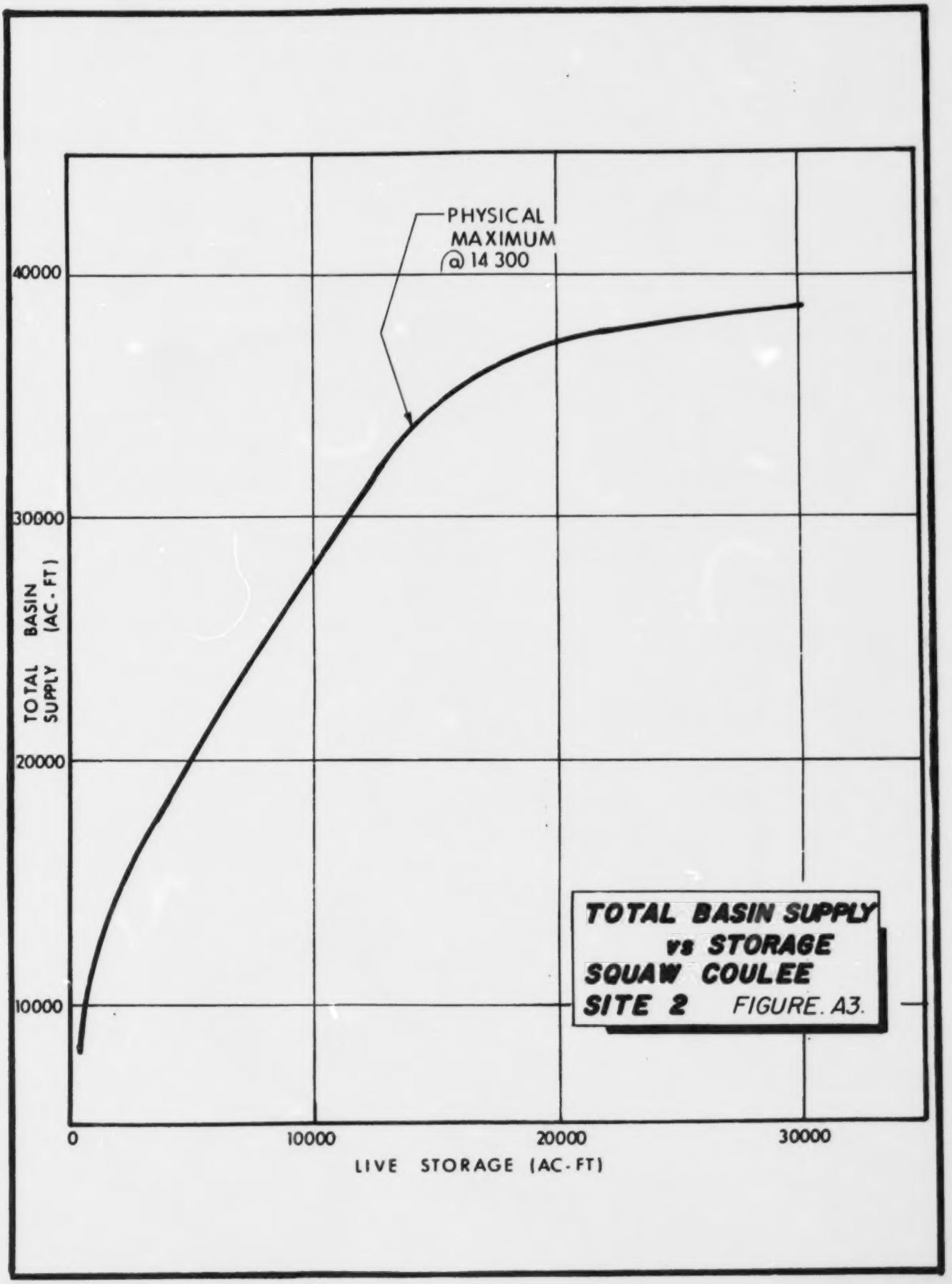
Little Bow #1	47,000 ac.ft.
Mosquito Creek #4	>32,000 ac.ft.
Squaw Coulee #2	>31,000 ac.ft.

The yield storage curves on Figures A1, A2 and A3 illustrate the reduction in yield as storage increases above 14,000 ac.ft. The hydrologic optimal storage occurs at the break-point of the straight line portions of the graphs. On this basis Squaw Coulee #2 is at the optimum while Little Bow #1 is slightly larger than optimum size and Mosquito Creek #4 is 30% larger than optimum.


PHYSICAL MAXIMUM @ 17 150
40000
30000
20000
10000
TOTAL BASIN SUPPLY (AC-FT)
0
10000
20000
30000
LIVE STORAGE (AC-FT)
TOTAL BASIN SUPPLY vs. STORAGE
LITTLE BOW SITE
FIGURE. A1.



TOTAL BASIN SUPPLY vs. STORAGE MOSQUITO CREEK SITE 4 FIGURE. A2.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40000
30000
20000
10000
TOTAL BASIN SUPPLY (AC-FT)
0
10000
20000
30000
LIVE STORAGE (AC-FT)
TOTAL BASIN SUPPLY vs STORAGE SQUAW COULEE SITE 2 FIGURE. A3.

2.0 FLOODS

The probable maximum floods (PMF) were determined for each site using the SNBB method for prairie streams. The 1 in 100 and 1 in 25 year floods for Mosquito Creek site and the sites on Squaw Coulee and Little Bow were determined using frequency analysis from different streams and independent methods of transposition. The Mosquito Creek basin was considered to be physiographically similar to Willow Creek whereas Squaw Coulee and the upper reach of the Little Bow were closer to a true prairie stream with intensive agricultural development. The two approaches to estimating flood flows are described below:

2.1 Mosquito Creek #4

The 1:100 and 1:25 year floods were estimated from frequency analysis of maximum mean daily discharges of Willow at Nolan and Claresholm. Pearson's Type III distribution was used to fit the logarithms of the recorded flood data. The peak instantaneous flow was calculated based on the average ratio of recorded peak to daily mean flow at Nolan and Claresholm. The floods were transposed to the Little Bow Basin using drainage area ratios in the formula

$$\frac{Q_{mosq}}{Q_{will}} = \left[\frac{A_{mosq}}{A_{will}}\right]^{n}$$

Where 'n' was determined from the 10 years of overlapping records of the Mosquito Creek at Nanton and Willow Creek at Nolan. The estimated values of flood flows are listed in Table 2.

TABLE 2

ESTIMATES OF FLOOD FLOWS

SITE	DRAINAGE AREA (mi^2)	PMF MAX. DAILY (cfs)	100 YEAR PEAK INSTANTANEOUS (cfs)	25 YEAR (cfs)
MOSQUITO CREEK #4	175	15750	9500	5000
SQUAW COULEE #2	20	5600	1200	600
LITTLE BOW #1	30	7200	1500	700

2.2 Squaw Coulee #2 and Little Bow #1

The 1:100 and 1:25 year flood flows were determined using two different techniques employing recorded flows of Nanton Creek near Nanton and West Arrowwood Creek near Arrowwood. This latter basin is adjacent, on the northeast side, of the upper reach of the Little Bow. Nanton Creek is a small watershed (120 km^2) with a flow record which overlaps the record of the Mosquito Creek at Nanton.

A frequency analysis of the recorded maximum daily discharges of West Arrowwood Creek using log Pearson Type III produced estimates of the 1:100 and 1:25 year flood flows. These were converted to peak instantaneous values using the average ratio of recorded peaks to daily discharges. The corresponding flood flows at Squaw coulee #3 and Little Bow #1 were then calculated using the same runoff per unit area as Arrowwood Creek. The values produced were considered to be minimum estimates of flood flows since the drainage area of West Arrowwood is approximately 10 times greater than the drainage area of the two reservoir sites.

Since Nanton Creek and Mosquito Creek have overlapping records of flow, floods on Nanton Creek can be related to Mosquito Creek through the drainage area ratio formula:

$$\frac{Q_{mosq}}{Q_{nant}} = \left[\frac{A_{mosq}}{A_{nant}}\right]^{1.35}$$

where the power term, 1.35, was determined from the four most severe floods on Nanton Creek. The 1:100 and 1:25 year floods of Nanton Creek were determined using this ratio. Since no drainage area relationship could be established

these flood values were considered to be maximum estimates for the two reservoir sites. The drainage area of the two sites is roughly one-half that of Nanton Creek.

The final values for the 1:100 and 1:50 year flood flows at Squaw Coulee #2 and Little Bow #1 were estimated as the average of the minimum and maximum values calculated above. The full results of the analysis of flood hydrology are displayed on Table 2.

LITTLE BOW RIVER BASIN
STORAGE SITE INVESTIGATION
FEASIBILITY DESIGN REPORT
June, 1985

APPENDIX B

GEOTECHNICAL EVALUATION

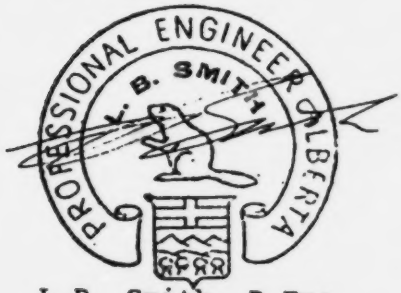
LITTLE BOW RIVER BASIN STUDY
GEOTECHNICAL EVALUATION

Submitted

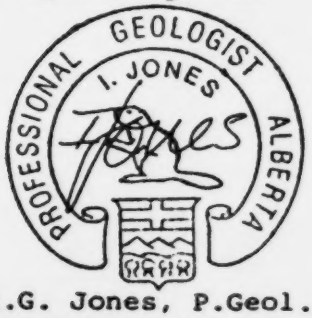
to

W-E-R ENGINEERING LTD.

G.C. Morgan, P.Eng.
Review Principal

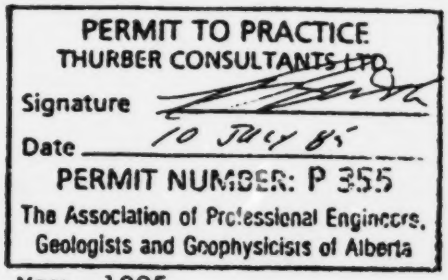


L.B. Smith, P.Eng.
Project Engineer



I.G. Jones, P.Geol.
Project Geologist

Thurber Consultants Ltd.
Calgary, Alberta


PERMIT TO PRACTICE
THURBER CONSULTANTS LTD.
Signature
Date 10 July 85
PERMIT NUMBER: P 355
The Association of Professional Engineers, Geologists and Geophysicists of Alberta

May, 1985
File: 19-293-0

SYNOPSIS

subject

This report presents a preliminary geotechnical evaluation of three proposed damsites located in the Little Bow River Basin, in the vicinity of High River, Alberta. The three sites discussed in this report are:

- Little Bow Site 1
- Mosquito Creek Site 4
- Squaw Coulee Site 2

field work

Comprehensive drilling programs were carried out at Little Bow River Site 1 and Mosquito Creek Site 4 by PRFA. Detailed information on subsurface conditions at each of these two sites is therefore available for the geotechnical evaluation. Three test holes were recently drilled in the vicinity of Squaw Coulee Site 2 by Thurber Consultants in order to permit a preliminary assessment of subsurface conditions at that site.

site geology

Geological conditions at all three sites are similar. On the uplands, silty clay till overlies bedrock at the sites to depths of 20 to 30 feet (6 to 9 m). The valleys are cut down into shale and sandstone bedrock which is covered on the valley walls by a thin veneer of colluvium (slopewash). Alluvial silty clays, silts, sands and gravels are present on the valley floor at all three sites. These materials are highly variable in their distribution and engineering properties. At Little Bow Site 1 and Squaw Coulee Site 2, the thickness of these alluvial sediments range up to 20 feet (6m). At Mosquito Creek Site 4, the sediments range up to about 60 feet (18 m) in thickness. The alluvium is underlain by shale and sandstone bedrock at all three sites.

reservoir shoreline stability

No significant concerns with respect to reservoir shoreline stability were identified for any of the three reservoirs. Deep seated landslides are possible in all cases, however the frequency of occurence of such

landslides is expected to be very low. Shallow instability can be expected along the reservoir shorelines due to wave erosion. The beaches are expected to be self stabilizing over 20 to 30 years.

reservoir watertightness

Lateral seepage from the reservoirs is not a major concern at any of the sites. Lateral seepage losses are expected to be greatest from the Squaw Coulee reservoir, because the full supply level of this reservoir is about 100 feet (30 m) above the regional groundwater table. A buried preglacial river valley intersects the Mosquito Creek Site 4 reservoir. It is speculated that seepage from the reservoir into this valley will not be significant; however, a detailed hydrogeological investigation should be undertaken to confirm this, before a final decision is made to construct a dam at this site.

embankment cross-section

At all three damsites a zoned earth fill should be used. It is recommended that the downstream slopes be constructed at an average 3 horizontal to 1 vertical, while the upstream slopes should be constructed at an average 4 horizontal to 1 vertical. It is anticipated that the slopes can be bermed in order to reduce the quantity of fill required for the embankments. A decision in this regard will depend on the results of material balance studies, laboratory test results and stability analyses which should be undertaken during detailed design.

seepage below embankment

Measures to control seepage below the dam embankments are essentially the same at Little Bow Site 1 and Squaw Coulee Site 2. At both sites, a compacted clay cutoff excavated through 15 to 20 feet (4.5 to 6 m) of pervious alluvium into bedrock is recommended. A conventional downstream drainage blanket and toe drain are also recommended at these two sites. More extensive seepage control measures are required at Mosquito Creek Site 4, because of the significant depth of alluvial

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sediments (60 feet) at this site. It is recommended that a sheet pile or slurry trench cutoff be installed to a depth of 40 feet (12 m) at this site to cut off the more pervious sediments. A drainage blanket and relief wells at the toe of the embankment are also recommended.

abutment seepage

Seepage around the abutments, through fractures and joints in the bedrock, is a concern at all three damsites. Special preparation of the abutments, as outlined in the report, will be required. It may be necessary to grout the rock at the abutments; however, further field investigations should be undertaken before a decision is made in this regard.

embankment cracking

Cracking of the embankment at Mosquito Creek Site 4, as a result of consolidation of the underlying alluvium, is a concern. Measures to mitigate the possible effects of cracking at this site include the use of an upstream zone of well graded gravelly silt, a chimney drain in the downstream portion of the embankment and compaction of the impervious clay fill at a water content which is wet of optimum. Such measures are not considered necessary at Little Bow Site 1 or at Squaw Coulee Site 2.

spillways

A single spillway, excavated around the dam abutment, is being proposed at all three of the sites. The spillway at Little Bow Site 1 will be excavated through about 15 feet (4.5 m) of clay till and 15 feet (4.5 m) into the underlying shale and sandstone bedrock. The excavation for the spillway at Mosquito Creek Site 4 will extend through about 25 feet (7.5 m) of clay till and about 5 feet (1.5 m) into the underlying bedrock. The spillway excavation at Squaw Coulee Site 2 will be made through several feet of colluvium and about 25 feet (7.5 m) into the underlying bedrock. No significant construction difficulties with any of these excavations are anticipated. Blasting of cemented sandstone layers will be required if such

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material is encountered within the bedrock. Permanent slopes in clay till should be cut at 3 horizontal to 1 vertical.Permanent slopes in the bedrock should be cut at 2 horizontal to 1 vertical, primarily to minimize surface erosion of the shale. If steeper slopes are desired in the bedrock, surface treatment, such as shotcrete, should be considered.

low level conduits

The low level conduits at all three sites should be founded either on bedrock, or on compacted silty clay fill which extends below the conduit down to the bedrock. The conduit should therefore be located where the depth to bedrock is a minimum at all three sites. In the case of Mosquito Creek Site 4, the preferred location is adjacent to the north abutment.

concrete

Sources for concrete aggregate have not been confirmed at any of the sites. It is anticipated that suitable aggregate (which will have to be crushed, screened and washed) can be located in the alluvial sediments upstream from Little Bow Site 1 and Mosquito Creek Site 4. No source for concrete in the immediate vicinity of Squaw Coulee Site 2 has been identified. It is recommended that Type 50 sulphate resistant cement be used for all concrete in contact with soil at all three sites.

additional geotechnical investigations

Prior to final design, a comprehensive drilling program will be required at Squaw Coulee Site 2. Such work is not necessary at Little Bow Site 1 or Mosquito Creek Site 4 since this work was carried out previously by PFRA. Supplementary geotechnical investigations and analyses would, however, be required at all three sites, as described in the report.

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SECTION 1

INTRODUCTION

1.1 General

This report presents the results of a geotechnical evaluation of three proposed dam sites located within the Little Bow River Basin. The damsites are designated as follows:

- Little Bow Site 1
- Mosquito Creek Site 4
- Squaw Coulee Site 2

1.2 Project Background

Eight potential damsites in the Little Bow River Basin were evaluated in connection with a water storage investigation study undertaken by W-E-R Engineering Ltd. for Alberta Environment Planning Division. The results of this evaluation (including a preliminary geotechnical evaluation) are presented in a conceptual design report which was submitted to Alberta Environment (W-E-R Engineering Ltd., 1985). The report identified the three potential sites, as listed above, which have the best overall potential for water storage within the Basin.

This report provides a geotechnical evaluation of these three sites. The scope of work for the geotechnical evaluation was established in discussions with Mr. Jack Wise, P.Eng. of W-E-R Engineering Ltd.

1.3 Report Organization

Sections 1, 2 and 3 of the report contain information which is generally applicable to all three of the sites included in the study.

A geotechnical evaluation and recommendations for Little Bow Site 1, Mosquito Creek Site 4 and Squaw Coulee Site 2 are presented in Section 4, 5 and 6 respectively. Each of these last three sections has been written so that each section can be read without reference to the remaining two sections. While this format has resulted in some repetition among these three sections, it was felt to be more convenient for future use.

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In accordance with the established report format for this study, Little Bow Site 1 and Mosquito Creek Site 4 are in Imperial units. Squaw Coulee Site 2 is in metric (S.I.) units.

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SECTION 2

METHOD OF INVESTIGATION

2.1 General

The geotechnical investigation carried out for the three sites consisted of the tasks outlined in the following sections.

2.2 Review Background Information

All available published and unpublished information concerning the geology and hydrogeology of the three sites was reviewed. A list of the sources of information referred to is presented at the end of the text of this report.

Geotechnical investigations were undertaken by PFRA at Little Bow Site 1, in 1968 and at Mosquito Creek Site 4 in 1967. These reports provided valuable information concerning the subsurface conditions at these two sites.

No previous subsurface investigations had been undertaken at Squaw Coulee Site 2 and only limited information was available at this site.

2.3 Engineering Terrain Analysis

An engineering terrain analysis of each of the three sites was carried out using high and low level airphoto coverage, to assess the geological setting and its implications on construction of the proposed dams.

The analysis included the mapping of surfical soils at each damsite and within the reservoir, identification of slope instability, and the delineation of surface drainage patterns and other pertinent features.

2.4 Field Drilling

One test hole (85-1) was drilled to a depth of about 22 metres at Little Bow Site 1. The purpose of this hole was to obtain core of the bedrock at the proposed site for detailed examination. This work was undertaken in order to examine the bedrock at this site for possible slickensides and other planes of weakness, which may not have been encountered in the investigation undertaken by PFRA.

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Three test holes (85-2, 3 and 4) were drilled to depths ranging from 6.5 to 18 metres in the vicinity of Squaw Coulee Site 2. These holes were drilled in order to obtain a preliminary indication of subsurface conditions at this site.

The four test holes were drilled during the period from 13 to 16 March, 1985 using an AR9000 drill rig operated by Mobile Augers and Research Ltd. of Edmonton. All test holes were filled to ground surface with cement grout on completion of drilling.

The log for the test hole drilled at Little Bow Site 1 is presented in Appendix A. The logs for the three holes drilled at Squaw Valley Site 2 are presented in Appendix C.

2.5 Laboratory Testing

Visual classifications and water content tests were carried out on all samples of the overburden soils recovered during the drilling program. The bedrock core was visually logged in the laboratory and index tests were carried out on selected representative samples of the core. The results of the laboratory tests are presented on the test hole logs.

SECTION 3

REGIONAL GEOLOGY

3.1 Regional Topography

The three damsites are situated within the Alberta Plain physiographic division of the Interior Plains. The locations of the three sites are shown on Figure 1.

As shown on Figure 1, the terrain within the area generally slopes gently from west to east. Elevations range from nearly 1100 m above sea level adjacent to the Porcupine Hills in the west to about 950 m above sea level close to the Little Bow River valley in the southeast. Superimposed on this gently sloping plain is a series of ice-marginal meltwater channels. These are incised to depths of 15 to 30 m below the general prairie level within the study area.

Relief is generally gently undulating to rolling, with local areas of subdued hummocky terrain. Slopes of less than 15 degrees are characteristic; however, slopes in excess of 50 degrees exist along the walls of the various meltwater channels.

3.2 Bedrock Geology

The Paskapoo Formation, of Upper Cretaceous to Tertiary age, is the uppermost bedrock unit within the study area. It comprises an interbedded sequence of non-marine shales, siltstones and medium to coarse-grained sandstones. Minor constituents are conglomerates and, rarely, bentonitic shale or bentonite seams. The bedrock is usually masked by younger Pleistocene and Holocene sediments. Locally, it is exposed, or present beneath a shallow colluvial veneer, along the walls of the various meltwater channels.

3.3 Buried Valleys

In pre-glacial times, an extensive network of river and stream valleys was developed within the study area. These were cut down into bedrock and have subsequently been infilled by a thick overburden sequence that may include a basal sand and gravel unit. Within the area of interest, the Silver Valley, the thalweg for which is indicated on Figure 1, is most important.

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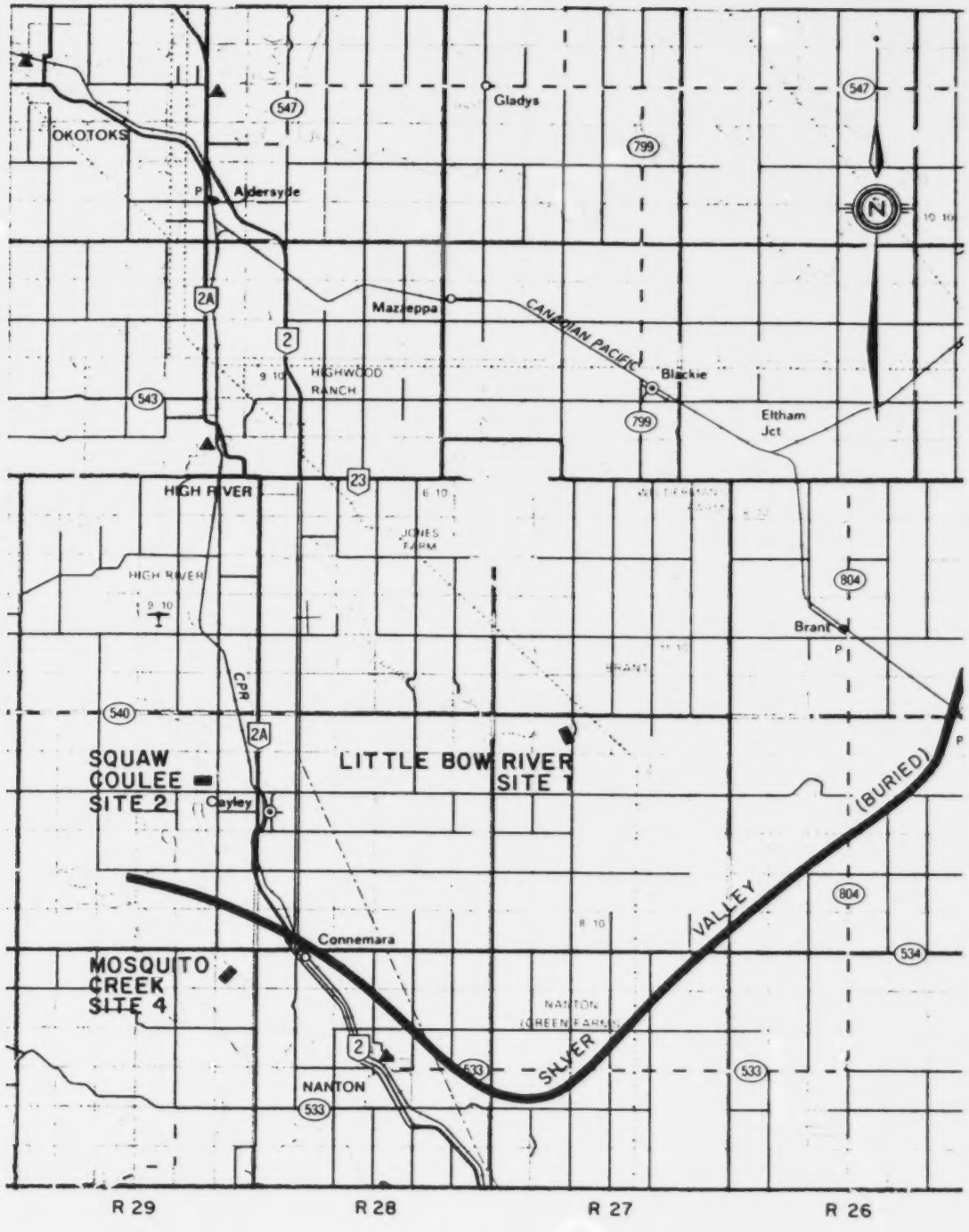

FIGURE 1. LOCATION OF PROPOSED DAMSITES

3.4 Surficial Geology

Glacial till, forming ground moraine and, locally, hummocky moraine, is the dominant surficial material within the study area. It comprises a heterogeneous mixture of clay to gravel size material, deposited either by Laurentide ice as it traversed the area (ground moraine) or from ice that decayed in place (hummocky moraine). Glaciofluvial deposits, sands and gravels predominantly, deposited by meltwater, are also locally abundant, forming esker, delta and kame deposits. In the southeast portion of the study area, silts and clays of glaciolacustrine (glacial lake) origin locally form a veneer over the morainal deposits.

Since deglaciation, post-glacial erosional and depositional processes have led to the development of a variety of more recent deposits. Materials of alluvial and colluvial origin are most important, respectively forming floodplain and terrace, and slopewash deposits along the floors and walls of the various meltwater channels.

The thickness of the surficial deposits overlying the bedrock averages in the order of 10 m over much of the study area. Within the buried pre-glacial valleys, however, considerably thicker surficial deposits may be expected.

3.5 Surface Drainage and Groundwater

The area of interest is drained by a series of generally northwest to southeast trending streams, of which the Little Bow River, Mosquito Creek and Squaw Coulee are amongst the most important. Surface drainage and run-off is generally into these water courses, but locally into closed depressions (such as Frank Lake), whence it either evaporates or seeps into the ground.

Three groundwater flow regimes, one superimposed on the other, are identified. On a regional scale, groundwater flows exist beneath the area at depth, from recharge areas in the Foothills and Rockies to discharge further east in the Plains. On a smaller scale, local groundwater flows are recharged in the higher sections of the area and discharge into the river and creek valleys. Finally, perched flows exist within granular, and more permeable, overburden zones and at the overburden-bedrock contact. These discharge into terrain depressions, as well as into river and creek valleys.

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SECTION 4

LITTLE BOW SITE 1

4.1 General

Little Bow Site 1 is located about 10 miles southeast of High River, Alberta, as shown on Figure 1. The damsite is located in the East Half of Section 32, Township 17, Range 27, West of the 4th Meridian.

The proposed reservoir would have a full supply level at 3290 feet which would result in a reservoir about 6 miles long. The dam embankment would have a maximum height of about 60 feet, a crest length of 1650 feet and a freeboard of 9 feet. The normal operating drawdown range would be about 40 feet.

A concrete chute spillway would be constructed around the left abutment of the embankment. This spillway would require an excavation of up to 30 feet below ground surface.

A low level conduit would be constructed through the embankment. The conduit would have an intake elevation at about 3245 feet and would be about 6 feet in diameter. Flows through the conduit would be controlled by gates accessed by a manhole immediately upstream of the centre-line of the embankment.

4.2 Reservoir Geology

The surficial geology of the reservoir is shown in Appendix A, Drawing 4.1. Ground moraine, consisting of silty clay, is the near suface soil on the uplands adjacent to this reach of the Little Bow River. As indicated on Drawing 4.1, the valley bottom is infilled with recent alluvial deposits, consisting of silts, sands and gravel.

The Little Bow River Valley has been cut down through the glacial till and into the underlying bedrock. Bedrock is exposed in some locations along the valley walls; however, in most locations it is covered by a thin veneer of colluvium (slopewash).

The valley walls are generally steep, ranging up to 45 degrees. No evidence of deep seated slope instability is apparent on the valley walls along this reach of the

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river. Shallow, localized instability of the colluvial materials occurs in a few locations, particularly where the Little Bow River is eroding directly at the toe of the valley walls.

4.3 Reservoir Shoreline Stability

Impoundment of the reservoir will cause a decrease in the overall stability of the valley walls mainly due to rapid drawdown and wave erosion. In addition, increased irrigation on the uplands adjacent to the reservoir would also tend to decrease the stability of the slopes. These effects may be significant, depending on local topography. However, the valley slopes should remain generally stable and deep seated large scale landslides are unlikely to occur along the major portion of the reservoir slopes.

The limits of any deep seated slope instability which does occur along the reservoir are expected to be confined within a zone described by a line drawn parallel to the valley at a distance of about 100 feet back from the top of the valley walls. This setback could be reviewed for site specific cases.

The reservoir full supply level is located below the elevation of the bedrock surface along this reach of the river. Therefore, the reservoir shoreline will erode into thin colluvium which is underlain by shale and sandstone bedrock. It is expected that small scale, localized instability of the slopes will occur due to wave erosion along the shoreline, particularly in locations where uncemented sandstone is present at the shoreline. This instability will be confined to the range of drawdown in the reservoir. The instability is expected to be self stabilizing as beaches develop over a period of 20 to 30 years.

4.4 Reservoir Watertightness

Detailed information concerning the present ground water elevation adjacent to the proposed reservoir is not available. Information from small scale hydrogeological maps for this area indicate, however, that the ground water table elevation is about 3300 feet. Assuming a reservoir elevation of 3290, it can be expected that the reservoir will not have a significant effect on the ground-water table in the area, except in a peripheral zone.

Some lateral seepage into or out of the valley walls will undoubtedly occur, primarily through joints and fractures in the bedrock, as well as through the more

pervious clean sandstone beds. However, in view of the anticipated existing high regional groundwater table, water leakage from the reservoir into the valley walls is not expected to be significant.

No buried valleys have been reported along this reach of the Little Bow River Valley, or in the vicinity of the damsite. Leakage into pervious zones within such valleys is therefore not identified as a concern at this time.

4.5 Damsite Geology

A preliminary geotechnical investigation of Little Bow Site 1 was undertaken by PRFA (1968). For convenience, a brief summary of the results of that investigation is presented here. More detailed information can be obtained from the report prepared by PFRA.

4.5.1 Surface Conditions

The distribution of surficial deposits and other pertinent features at Little Bow Site 1 is presented on Drawing 4.2 in Appendix A.

The near surface soils on the uplands in the vicinity of the damsite are a morainal deposit consisting of a silty clay.

The valley slopes near the damsite are covered by a thin veneer of colluvial material over bedrock. The valley slopes are moderately steep, ranging from about 32o (about 1.5:1) at the south abutment to about 16o (about 3.5:1) at the north abutment. No evidence of deep seated instability of the valley walls is apparent in the vicinity of the damsite.

The valley bottom is relatively flat and has a mantle of recent alluvial deposits overlying bedrock. Little Bow River is an underfit stream, occupying an early post-glacial meltwater channel.

As indicated on Drawing 4.2, a tributary stream flows into the Little Bow River immediately downstream from the proposed damsite. Deltaic deposits, likely consisting of silts and sands have been deposited by this tributary on the north side of the Little Bow River valley, as shown on the drawing.

4.5.2 Subsurface Conditions

Subsurface conditions at the damsite are presented in Drawing 4.3. This drawing is a simplified version of a detailed subsurface cross-section at the damsite presented in the PFRA report. The following major soil units were encountered at the site:

- clay till
- alluvium
- bedrock

Clay Till

Silty clay till was found to overlie the bedrock on the uplands on both sides of the damsite. The clay till ranges up to 23 feet in thickness in the immediate vicinity of the damsite. Its thickness can be expected to increase at locations further from the top of the valley slope. The clay till is very stiff to hard in consistency and has a low to medium plasticity. Average properties of the clay till are summarized on Table 4.1.

Alluvium

Alluvial materials were encountered in the valley bottom, as indicated on Drawing 4.3. These materials were found to be highly variable in texture, ranging from silty clays to clean sands and gravels. In general, the texture of the alluvial sediments becomes coarser (and the materials more pervious) with depth. The alluvial sediments were found to range up to 17 feet in depth at the damsite.

The cohesive alluvial deposits ranged from firm to very stiff in consistency. The penetration resistance of the cohesionless deposits was not reported; however, it is expected that the density of these materials will range from loose to compact.

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Table 4.1

Little Bow Site 1
Summary of Soil Properties

Property	Unit	Silty Clay Till	Shale Bedrock	Sandstone Bedrock
Wet Density	PCF	127	135	151
Water Content	%	13	12	5
Liquid Limit	%	35	41	27
Plastic Limit	%	15	21	21
Plasticity Index	%	20	20	6
Undrained, Unconfined Shear Strength	psi	26	225	1060
Activity	--	--	0.34	--
Modulus of Elasticity	psi	--	15,000	110,000
Water Content at Standard Proctor Maximum Density	%	15.5	--	--

NOTES: 1) Values for soil properties given above are average values as reported by PFRA (1968).

2) The value reported for the modulus of elasticity of the shale bedrock appears to be unreasonably low.

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Bedrock

Bedrock consists of interbedded shales and sandstones. The bedrock was classified by PFRA as shale or sandstone depending on its dominant properties. As discussed by PFRA, the bedrock includes the entire spectrum from shale, sandy shale, silty shale, shaly sandstone to sandstone. The various lithologies grade from one to another, making a more specific classification difficult.

The average properties of the shale, as reported by PFRA, are summarized in Table 4.1. The shale at this site can be classified as a weak rock. It can be expected to weather rapidly to a medium to high plastic clay when left exposed at ground surface.

No slickensides or other planes of weakness were noted in any of the core taken from the recent drill hole. A bentonite seam was noted in one test hole drilled by PFRA at this site. The activity of the clay shale, as reported by PFRA, is relatively low, indicating that montmorillinite does not form a significant proportion of the clay minerals within the shale. Montmorillinite has a low angle of friction compared to other clay minerals and is often found in those rocks which are prone to instability. Most significantly, no deep seated bedrock failures have been observed along the valley slopes upstream and downstream of the proposed damsite. Such instability is common in locations where pre-existing failure surfaces are found in the Cretaceous sediments of Western Canada. On the basis of the foregoing information, it has been concluded that peak strength parameters (rather than much lower residual strength parameters) will be acceptable for use in design of the dam embankment and other structures at this location.

The properties of the cemented sandstone at this site are also presented on Table 4.1. The sandstone can be classified as a moderately strong rock. Most sandstones are clay and calcite cemented. The cemented sandstones were found to break down when exposed to water for several days. The distribution of the stronger, cemented, sandstones has not been clearly established on the basis of the information provided by PFRA.

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Groundwater

No seepage zones or free water was encountered in any of the test holes drilled by PFRA on the uplands adjacent to the damsite. Water levels in the alluvial deposits on the valley floor are assumed to be at about the same elevation as river level. No artesian water pressures were reported in any of the test holes drilled by PFRA into bedrock at this site.

Pneumatic piezometers were not installed by PFRA and hence piezometric elevations within the impervious bedrock formations at this site are not known. A detailed hydrogeological assessment of the site has not been undertaken.

4.6 Dam Embankment

4.6.1 Siting

There are no significant geotechnical factors affecting the positioning of the embankment centreline at this site. It should be recognized however, that if the embankment is moved from the location proposed by PFRA, it will be necessary to drill additional test holes on the new embankment centreline. It is possible, but not likely, that subsurface conditions at any new location may be significantly different from those encountered by PFRA.

A small ravine is located on the south side of the valley a short distance upstream from the centreline proposed by PFRA, as shown on Drawing 4.2. If the embankment centreline is moved further upstream, measures may be required to prevent surface water from this ravine from eroding the downstream face of the embankment.

4.6.2 Embankment Configuration

A tentative configuration for the proposed dam embankment is presented in Appendix A, Drawing 4.4. It is expected that the configuration presented on Drawing 4.4 would be refined and optimized during detailed design, as a result of material balance studies and detailed slope stability analyses.

An average upstream slope of 4 horizontal to 1 vertical is recommended, with an average downstream slope of 3 horizontal to 1 vertical. Both the up-

stream and downstream slopes may be bermed in order to reduce the volume of material required. A decision in this regard would be made during the detailed design phase.

The upstream portion of the dam (Zone 1) would be constructed of medium plastic silty clay till. This material would be obtained from the upper portion of the spillway excavation. Additional borrow sources for the silty clay till would probably have to be located on the uplands, since it is unlikely that sufficient material would be liberated from the spillway excavation. The silty clay till is 2 or 3% dry of optimum water content and it will be necessary to add water during compaction.

The downstream portion of the dam (Zone 2/3) would be constructed of random semi-pervious to pervious sands and gravels taken initially from the cutoff excavation. Additional material for this portion of the dam would have to come from borrow sources located on the valley floor, upstream from the dam. The coarser and more pervious materials should be placed towards the downstream face of the dam.

Compacted shale and sandstone bedrock from the spillway excavations could also be used in the downstream portion of the dam as indicated on the drawing. As noted, the exact boundaries of the various zones can be modified to make optimum use of the available materials.

Clean sand and gravel would be placed to form the toe drain, as shown on the drawing. This material may be available within the alluvium, however it may be necessary to screen and wash the pit run gravels in order to obtain material meeting the required specifications. Filter cloth could be used, as indicated on the drawing, to prevent migration of fines into the toe drain.

Preliminary calculations indicate that the dam foundation can be expected to settle in the order of 1 foot or so during construction. The dam crest would settle an additional 6 inches or so when the reservoir is filled. It may be desirable to raise the finished dam crest above the design elevation in order to accommodate movements of this order. If required, a detailed settlement analysis of the embankment can be undertaken during detailed design.

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4.6.3 Seepage Control

All organic material and all soft or loose inorganic materials should be stripped from below the area of the embankments. The entire area should be scarified and recompacted to a depth of at least 1 foot. Proof rolling is recommended prior to placing fill in order to identify any soft zones. Waste soil should be placed upstream of the dam preferrably in the existing creek channel.

Pervious layers within the alluvium represent the most serious source of seepage beneath the embankment. A positive cutoff, extending into the shale bedrock below the embankment is recommended. The width of the base of the cutoff could vary from a minimum 30 feet to 50 feet (or greater). The width of the cutoff could be increased, as desired, in order to obtain the required quantities of gravel for use in the downstream toe drain. Seepage into the cutoff excavation and the stability of the slopes of the cutoff excavation are a concern. It is recommended that a number of test pits be excavated along the centreline of the cutoff in order to provide a better assessment of the behaviour of excavation sideslopes.

Seepage through fractures and joints in the bedrock below the dam is a concern. It may be necessary to extend the toe drain down into the bedrock in order to relieve potential uplift pressures at that point. Alternatively, pressure relief wells should also be considered. The requirement in this regard will depend on the bedrock stratigraphy below the downstream toe.

Abutment seepage is a major concern. The precise requirements to control this source of seepage cannot be made until data from additional field permeability tests in the bedrock is obtained. It may be necessary to inject grout into the abutments in order to control seepage. The bedrock at the abutments of the dam would likely be stepped, in order to facilitate the placement and compaction of fill against the abutments. Seepage along the interface between the compacted clay fill and the bedrock may require shotcreting of the bedrock surface in that area where the impervious clay core is placed against the abutment. Silty clay compacted against the abutments should be compacted at 2 or 3% wet of

optimum. Further, a clean gravel layer should be placed against the bedrock surface immediately downstream from the impervious zone. This gravel layer should be connected to a pipe and, if practical, discharged to a location where it can be monitored. The purpose of the gravel layer is to safely drain any seepage water from the bedrock and the bedrock contact.

4.6.4 Upstream Slope Protection

The upstream face of the dam must be armoured within the normal operating reservoir levels in order to reduce possible wave erosion. As indicated on Drawing 4.4, the slope protection should consist of 1.5 feet of well graded sand and gravel covered by 2 feet of rock armour. The class of rock armour required will be established during detailed design.

A suitable source of rip rap slope protection has not been established. The local cemented sandstone is unsuitable for rip rap because it breaks down when exposed to water for prolonged periods.

An often acceptable alternative to expensive rip rap is to use coarse pit run gravel and accept the need for occasional regrading of the slopes. The use of soil cement could also be considered.

4.6.5 Downstream Slope Protection

Measures will be required to prevent erosion of the downstream face of the embankment due to surface runoff. The crest of the dam should be surfaced with gravel (for trafficability) and sloped towards the reservoir. The downstream face of the embankment should be covered with 1 foot of topsoil and hydroseeded.

4.7 Spillway

Construction of the spillway approach channel around the left abutment will require an excavation to a depth of about 30 feet below ground surface. The excavation will extend through about 15 feet of silty clay till and about 15 feet into bedrock. At the proposed spillway location, the bedrock consists primarily of shale, however some hard sandstone layers were encountered in the test holes drilled at the spillway location. No major water bearing layers were encountered within the proposed depth of the spillway excavation.

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No major difficulties with the spillway excavation are anticipated. It is expected that the shale and the weaker sandstones can be removed by ripping. Some blasting of cemented sandstone layers may be required. Occasional seepage zones may be encountered, however water quantities are expected to be low and water can be collected into drainage ditches and sumps using conventional construction practice.

Permanent cutslopes in the clay till should be no steeper than 3 horizontal to 1 vertical in order to ensure long term stability. Flatter sideslopes within the clay till may be desireable in order to liberate more material for use in the construction of the embankment.

Permanent cutslopes within the shale or sandstone bedrock could be trimmed at 1 horizontal to 1 vertical or steeper and they would remain stable. However, the exposed shale is susceptible to weathering and erosion which could become a major maintenance problem. For this reason, it is recommended that permanent slopes in the bedrock be cut no steeper than 2 horizontal to 1 vertical in order to minimize surface erosion. If steeper slopes are desired, surface treatment, such as shotcreting, should be considered.

It is understood that the spillway chute will be lined with concrete in order to prevent erosion. It is understood that the gradient for the spillway chute cannot exceed 3 horizontal to 1 vertical for hydraulic reasons. This slope will be satisfactory in the shale bedrock from a slope stability point of view. If necessary, steeper longitudinal slopes could be considered. In any event, it will be necessary to anchor the spillway chute to the slope, and install suitable drainage below the chute to prevent uplift due to water pressure.

It is understood that the concrete weir and gate structure at the top of the spillway chute will be subjected to horizontal loads, which must be transferred to the underlying shale bedrock. The horizontal shearing resistance of the shale bedrock will be less than the shear strengths reported by PFRA because of the presence of bedding planes parallel to the direction of shear. For preliminary design of the weir, a nominal angle of friction of 20 degrees may be used. This value should be confirmed during detailed design.

The shale bedrock can be expected to rebound due the weight of material removed from the excavation. Rebound will be primarily elastic and will therefore take place as the excavation proceeds. Past experience with deep excavations in materials similar to those encountered here, indicates that all rebound (including time dependent rebound) will be essentially complete 2 to 3 months after the excavation is completed. The total rebound will range from 1 to 3 inches. A more precise estimate of the rebound can be made once the configuration and depth of the excavation has been confirmed.

4.8 Low Level Conduit

In view of the highly variable compressibility of the alluvial sediments at this site, it is recommended that all alluvial materials below the low level conduit through Zone 1 be subexcavated down to bedrock, and replaced with well compacted silty clay. It is therefore desirable to place the conduit in a location where the depth of bedrock is at a minimum. The subsurface information reported by PFRA indicates that the most suitable location is in the vicinity of Boreholes 26 and 28.

It is recommended that cutoff collars be cast around the upstream portion of the conduit in Zone 1 in order to reduce the possibility of seepage. The downstream portion of the conduit through Zone 2/3 should be surrounded by a minimum thickness of 3 feet of pervious filter in order to prevent possible erosion along the conduit.

Settlement and horizontal elongation of the conduit can be expected as the embankment fill is constructed. The conduit must be designed to accommodate these movements. The deformations could be minimized if the conduit were placed on the bedrock near one of the abutments.

4.9 Concrete

A suitable source of concrete aggregate has not been identified. The alluvial deposits in the valley would have to be crushed, screened and washed in order to obtain suitable aggregate for concrete. The durability and soundness of these aggregates is not known. A number of water soluble sulphate tests were undertaken by PFRA and these indicate negligible water soluble sulphates are present in the soils at this site. These test results may not, however, be representative of the

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soils adjacent to concrete structures. Unless further water soluble sulphate tests are undertaken, it is recommended that Type 50 sulphate resistant cement be used for all concrete in contact with soil at this site. This is particularly important for the low level conduit, which is subject to an unfavourable hydraulic environment from the point of view of potential attack by sulphates.

4.10 Additional Geotechnical Investigations

If a decision is made to proceed with construction of a dam at Little Bow Site 1, the following geotechnical work is recommended:

1) Test pits should be excavated in the alluvial deposits along the dam centreline and upstream of the dam. The objectives of this work are:

   - To identify the quality and quantity of granular materials for use in embankment construction,
   - to locate and test suitable sources of concrete aggregate,
   - to establish the depth to bedrock along the centreline of the proposed cutoff, as well as to establish potential difficulties in making the excavation for the cutoff,
   - to establish the depth to bedrock along the centreline of the proposed low level conduit, as well as to establish potential construction difficulties, and
   - to establish the depth to bedrock along the centreline of the proposed toe drain.

2) High pressure injection tests should be undertaken in the bedrock, at the abutments, along the dam centreline and near the toe of the embankment, in order to assess the stratigraphy and permeability of the bedrock at these locations.

3) Once the location and depth of the spillway has been confirmed, additional test holes should be drilled along the proposed centreline. The purpose of these holes is:

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- To confirm subsurface conditions along the spillway centreline and in particular to establish the volume of cemented sandstone which must be blasted,

- to confirm groundwater conditions along the spillway centreline, and

- to sample and test the shale bedrock in order to assess the potential use of this material in embankment construction.

4) Additional test pits and test holes should be drilled in potential borrow areas for clay till. Specifically, it is necessary to determine the following:

   - The susceptibility of the clay till to internal piping (dispersion tests are recommended),

   - the water soluble sulphate content of the clay borrow, and

   - the compaction properties of the clay borrow.

5) It is recommended that pneumatic piezometers be installed at selected locations in the vicinity of the damsite in order to establish current piezometric pressures in the bedrock at the site. This information is required in order to evaluate the stability of the proposed embankment.

6) A program of laboratory strength testing should be undertaken on samples of compacted silty clay till, on undisturbed alluvium and on undisturbed samples of bedrock core in order to establish the effective stress strength and pore pressure parameters of these materials for use in stability analyses.

7) A suitable source of rip rap for upstream slope protection should be located.

8) An earth materials balance study should be carried out in order to optimize the configuration of the dam embankment as well as to establish the most efficient sequence of construction.

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9) Selective stability analyses should be undertaken in order to refine the design of the embankment.

10) Detailed field checking of the valley slopes along the reservoir should be undertaken in order to identify localized areas where the potential for instability is of concern.

11) A detailed hydrogeological assessment of the damsite and reservoir should be undertaken in order to assess the impact of the proposed reservoir on the groundwater regime. This work should include a more detailed evaluation of lateral seepage from the reservoir.

It is expected that 2 to 3 months would be required to complete the geotechnical investigations outlined above.

SECTION 5

MOSQUITO CREEK SITE 4

5.1 General

Mosquito Creek Site 4 is located about 13 miles south of High River, Alberta, as shown on Figure 1. The damsite is located in the Southeast Quarter of Section 36, Township 16, Range 29, West of the 4th Meridian.

The proposed reservoir would have a full supply level at 3423 feet which would result in a reservoir about 4 miles long. The dam embankment would have a maximum height of about 65 feet, a crest length of about 1050 feet, and a freeboard of 10 feet. The normal operating drawdown range would be about 50 feet.

A concrete chute spillway would be constructed around the left abutment of the embankment. This spillway would require an excavation of up to 30 feet below ground surface.

A low level conduit would be constructed through the embankment. The conduit would have an intake elevation at about 3378 feet and would be about 6 feet in diameter. Flows through the conduit would be controlled by gates accessed by a manhole immediately upstream of the centreline of the embankment.

5.2 Reservoir Geology

The surficial geology of the reservoir is shown in Appendix B, Drawing 5.1. Ground moraine, consisting of silty clay is the near suface soil on the uplands adjacent to this reach of Mosquito Creek. As indicated on Drawing 5.1, the valley bottom is infilled with recent alluvial deposits, consisting of silts, sands and gravel.

The Mosquito Creek Valley has been cut down through the glacial till and into the underlying bedrock. Bedrock is exposed in some locations along the valley walls, however in most locations it is covered by a thin veneer of colluvium.

The valley walls are moderately steep, ranging up to 35 degrees. No evidence of deep seated slope instability is apparent on the valley walls along this reach of the

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creek. Shallow, localized instability of the colluvial materials occurs in a few locations, particularly where Mosquito Creek is eroding directly at the toe of the valley walls. One such shallow landslide is located about 300 metres upstream from the proposed damsite on the right valley wall.

5.3 Reservoir Shoreline Stability

Impoundment of the reservoir will cause a decrease in the overall stability of the valley walls mainly due to rapid drawdown and wave erosion. In addition, increased irrigation on the uplands adjacent to the reservoir would also tend to decrease the stability of the slopes. These effects may be significant, depending on local topography. However, the valley slopes should remain generally stable and deep seated large scale landslides are unlikely to occur along the major portion of the reservoir slopes.

The limits of any deep seated slope instability which does occur along the reservoir are expected to be confined within a zone described by a line drawn parallel to the valley at a distance of about 100 feet back from the top of the valley walls. This could be reviewed for site specific cases.

The reservoir full supply level is located slightly above the contact between the silty clay till and the bedrock surface along this reach of the river. Therefore, the reservoir shoreline will erode into thin colluvium which is underlain by clay till or by shale and sandstone bedrock. It is expected that small scale, localized instability of the slopes will occur due to wave erosion along the shoreline, particularly in locations where sand is present at the shoreline. The instability will be confined to the range of drawdown in the reservoir. The instability is expected to be self stabilizing as beaches develop over a period of 20 to 30 years.

5.4 Reservoir Watertightness

Detailed information concerning the present groundwater elevations adjacent to the proposed reservoir is not available. Information from small scale hydrogeological maps indicates, however, that the average ground water table elevation in this area is about 3400 feet. Assuming a reservoir elevation of 3423 feet, it can be expected that the reservoir will cause a modest rise in the groundwater table in the immediate area.

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Some lateral seepage out of the valley walls will undoubtedly occur, primarily through joints and fractures in the bedrock, as well as through the more pervious clean sandstone beds. However, in view of the anticipated existing high regional groundwater table, seepage losses into the bedrock are not expected to be significant.

A buried valley (Silver Valley) has been mapped upstream of the proposed damsite (Geiger, 1968). The approximate location of this valley is shown on Figure 1. No information concerning the nature of the valley infill at its intersection with the proposed reservoir is available. Assuming the groundwater table within the preglacial valley is around 3400 feet, it is speculated that no significant seepage losses into the valley will occur. It is recommended, however, that a detailed subsurface investigation be undertaken in order to establish the stratigraphy within the buried valley, before a final decision is made to construct a dam at this site.

5.5 Damsite Geology

A preliminary geotechnical investigation of Mosquito Creek Site 4 was undertaken by PRFA (1967). For convenience, a brief summary of the results of that investigation is presented here. More detailed information can be obtained from the report prepared by PFRA.

5.5.1 Surface Conditions

The distribution of surficial deposit and other pertinent features at Mosquito Creek Site 4 is shown on Drawing 5.2, presented in Appendix B.

The near surface soils on the uplands in the vicinity of the damsite are morainal deposits consisting of silty clay.

The valley slopes near the damsite are covered by a thin veneer of colluvial material over bedrock. The valley slopes are moderately steep ranging from about 30o (about 2:1) at the south abutment to about 12o (about 5:1) at the north abutment. No evidence of deep seated instability of the valley walls is apparent in the vicinity of the damsite. As mentioned earlier, a shallow landslide in colluvium is located on the right valley wall about 300 m upstream from the damsite.

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The valley bottom is relatively flat and is underlain by a mantle of recent alluvial deposits overlying bedrock. Mosquito Creek is an underfit stream occupying an early post-glacial spillway or meltwater channel.

5.5.2 Subsurface Conditions

Subsurface conditions at the damsite are presented in Drawing 5.3. This drawing is a simplified version of a detailed subsurface cross-section at the damsite presented in the PFRA report. The following major soil units were encountered at the site:

- clay till
- alluvium
- bedrock

Clay Till

Silty clay till was found to overlie the bedrock on the uplands on both sides of the damsite. The clay till ranges up to 30 feet in thickness in the immediate vicinity of the damsite. Its thickness can be expected to increase at locations further from the top of the valley slope. The clay till is very stiff to very hard in consistency and has a low to medium plasticity. Average properties of the clay till are summarized on Table 5.1.

Alluvium

Alluvial materials were encountered in the valley bottom, as indicated on Drawing 5.3. These materials were found to be highly variable in texture, ranging from silty clays to clean sands and gravels. Alluvial sediments up to 61 feet in thickness were encountered in test holes put down at the damsite.

The cohesive alluvial deposits ranged from stiff to very stiff in consistency. The penetration resistance of the cohesionless deposits are not reported, however, it is expected that the density of these materials will range from loose to compact.

Bedrock

The bedrock at Mosquito Creek Site 4 consists of interbedded shales and sandstones as indicated on Drawing 5.3. The bedrock was classified by PFRA as shale or sandstone depending on its dominant properties. As discussed by PFRA, the bedrock covers the entire spectrum from shale, sandy shale, silty shale, shaly sandstone to sandstone. The various lithologies grade from one to another, making a more specific classification difficult.

The average properties of the shale, as reported by PFRA, are summarized in Table 5.1. The shale at this site can be classified as a moderately weak rock. It can be expected to weather rapidly to a medium to high plastic clay when left exposed at the ground surface.

No deep seated bedrock failures have been observed along the valley slopes upstream and downstream of the proposed damsite. Instability of this type is common in locations where pre-existing failure surfaces are found in the Cretaceous sediments of Western Canada. On the basis of the foregoing information, it is concluded that peak strength parameters (rather than much lower residual strength parameters) will be acceptable for use in design of the dam embankment and other structures at this location.

The properties of the cemented sandstone at this site are also presented on Table 5.1. The sandstone can be classified as a moderately strong rock. The stronger sandstones are clay and calcite cemented. The cemented sandstones are expected to break down if exposed to water for prolonged periods. The distribution of the stronger cemented sandstones has not been clearly identified on the information provided by PFRA.

Groundwater

No seepage zones or free water was encountered in any of the test holes drilled by PFRA on the uplands adjacent to the damsite. Water levels in the

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Table 5.1

Mosquito Creek Site 4
Summary of Soil Properties

Property	Unit	Silty Clay Till	Shale Bedrock	Sandstone Bedrock
Wet Density	PCF	132	142	150
Water Content	%	14	13	6
Liquid Limit	%	33	32	25
Plastic Limit	%	15	21	20
Plasticity Index	%	18	11	5
Undrained, Unconfined Shear Strength	psi	--	875	1900
Water Content at Standard Proctor Maximum Density	%	14.0	--	--

NOTE: Values for soil properties given above are average values as reported by PFRA (1967).

alluvial deposits on the valley floor are assumed to be at about the same elevation as creek level. No artesian water pressures were reported in any of the test holes drilled by PFRA into bedrock at this site.

Pneumatic piezometers were not installed by PFRA and hence piezometric elevations within the impervious bedrock formations at this site are not known. A detailed hydrogeological assessment of the site has not been undertaken.

5.6 Dam Embankment

5.6.1 Siting

There are no significant geotechnical factors affecting the positioning of the embankment centre-line at this site. It should be recognized however, that if the embankment is moved from the location proposed by PFRA, it will be necessary to drill additional test holes on the new embankment centre-line. It is possible, but not likely, that subsurface conditions at any new location may be significantly different from those encountered by PFRA.

5.6.2 Embankment Configuration

A tentative configuration for the proposed dam embankment is presented in Appendix B, Drawing 5.4. It is expected that the configuration presented on Drawing 5.4 would be refined and optimized during detailed design, as a result of material balance studies and detailed slope stability analyses.

An average upstream slope of 4 horizontal to 1 vertical is recommended, with an average downstream slope of 3 horizontal to 1 vertical. Both the up-stream and downstream slopes may be bermed in order to reduce the volume of material required. A decision in this regard would be made during the detailed design phase.

The upstream portion of the dam (Zone 1) would be constructed of medium plastic silty clay till. This material would be obtained from the upper portion of the spillway excavations. Additional borrow sources for the silty clay till would probably have to be located on the uplands, since it is unlikely that sufficient material would be liberated from the

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spillway excavations. The silty clay till is near optimum water content, however it will be necessary to add some water during construction since it is desireable to compact the clay at 2 to 3 percent above optimum in order to reduce cracking of the embankment as the underlying alluvium consolidates.

The downstream portion of the dam (Zone 2/3) would be constructed of random semipervious to pervious sands and gravels taken initially from the cutoff excavation. Additional material for this portion of the dam would have to come from borrow sources located on the valley floor. It is recommended that such borrow sources be established downstream from the damsite, in order to preserve the integrity of the near surface fine grained sediments on the valley floor upstream from the dam. A kame deposit located about 0.5 miles downstream from the dam also offers a possible source of granular materials. The coarser and more pervious materials should be placed towards the downstream face of the dam.

Compacted shale and sandstone bedrock from the spillway excavations could also be used in the downstream portion of the dam as indicated on the drawing. As noted, the exact boundaries of the various zones can be modified to make optimum use of the available materials.

Clean sand and gravel would be placed to form the toe drain, as shown on the drawing. This material may be available within the alluvium, however it may be necessary to screen and wash the pit run gravels in order to obtain material meeting the required specifications. Filter cloth could be used, as indicated on the drawing, to prevent migration of fines into the toe drain.

Significant settlements of the centre of the embankment structure can be expected due to the thickness of the alluvial deposits. Preliminary calculations indicate that total settlement of the embankment could range from 2 to 3 feet. A major portion of the settlement would occur during construction with settlement essentially complete several years following completion of the embankment. An additional foot of settlement (due to settlement of the compacted clay fill) is expected when the reservoir is filled. If required, a more compre-

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hensive settlement analysis of the embankment can be undertaken during detailed design. The crest of the dam should be cambered about 3 feet above the design elevation in order to accommodate the expected settlement.

The foundation settlements at this site are expected to cause cracking of the compacted silty clay. In order to mitigate this problem, the following measures are recommended:

- The clay core should be compacted at 2 to 3 percent wet of optimum water content.

- A gravelly silt layer about 10 feet in thickness should be placed against the upstream face of the dam as shown on Drawing 5.4. The gravelly silt will be drawn into any cracks by seepage, helping to seal the cracks.

- A pervious chimney drain should be constructed in the downstream portion of the dam as shown on the drawing. This drain must be designed to be suitably filtered.

5.6.3 Seepage Control

All organic material and soft or loose inorganic materials should be stripped from below the area of the embankment. The entire area should be scarified and recompacted to a depth of at least 1 foot. Proof rolling is recommended prior to placing fill in order to identify any soft zones. Waste soil should be placed upstream of the dam preferrably in the existing creek channel.

Pervious layers within the alluvium represent the most serious source of seepage beneath the embankment. A cutoff, extending at least 40 feet (El. 3340) below the base of the embankment is recommended in order to cut off the more pervious sediments. The cutoff could consist of either a steel sheet pile wall or a cement/bentonite slurry trench wall. The steel sheet pile wall has a major disadvantage in that is is susceptible to corrosion. The portion of the cutoff above the water table could consist of compacted clay, as indicated on the drawing.

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Although some seepage will occur, through the alluvium below the bottom of the cutoff, it will still greatly reduce the total amount of seepage below the dam, because the average vertical permeability of the alluvium is much less than its average horizontal permeability. Nevertheless, it is recommended that pressure relief wells be installed at the toe of the embankment to control seepage through the alluvium at depth. These wells should extend down to the bedrock surface. Consideration should also be given during detailed design to extending the cutoff entirely through the alluvial sediments into the underlying bedrock.

Abutment seepage is a major concern. The precise requirements to control this source of seepage cannot be made until data from additional field permeability tests in the bedrock are obtained. It may be necessary to inject grout into the abutments in order to control seepage. The bedrock at the abutments of the dam would likely be stepped in order to facilitate the placement and compaction of fill against the abutments. Seepage along the interface between the compacted clay fill and the bedrock may require shotcreting of the bedrock surface in that area where the impervious clay core is placed against the abutment. Silty clay compacted against the abutments should be compacted at 2 or 3% wet of optimum. Further, a clean gravel layer should be placed against the bedrock surface immediately downstream from the impervious zone. This gravel layer should be connected to a pipe, and if practical, discharged to a location where it can be monitored. The purpose of the gravel layer is to safely drain any seepage water from the bedrock and the bedrock contact.

5.6.4 Upstream Slope Protection

The upstream face of the dam must be armoured within the normal operating reservoir levels in order to reduce possible wave erosion. As indicated on Drawing 5.4, the slope protection should consist of 1.5 feet of well graded sand and gravel covered by 2 feet of rock armour. The class of rock armour required will be established during detailed design.

A suitable source of rip rap slope protection has not been established. The local standstone is unsuitable for rip rap because it is expected to break down when exposed to water for prolonged periods.

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An often acceptable alternative to expensive rip rap is to use coarse pit run gravel and accept the need for occasional regrading of the slopes. The use of soil cement could also be considered.

5.6.5 Downstream Slope Protection

Measures will be required to prevent erosion of the downstream face of the embankment due to surface runoff. The crest of the dam should be surfaced with gravel (for trafficability) and sloped towards the reservoir. The downstream face of the embankment should be covered with 1 foot of topsoil and hydro-seeded.

5.7 Spillway

Construction of the chute spillway around the left abutment will require an excavation to a depth of about 30 feet below ground surface. The excavation will extend through about 25 feet of silty clay till and about 5 feet into bedrock. At the proposed spillway location, the bedrock consists primarily of shale, however some hard sandstone layers were encountered in the test holes drilled at the spillway location. No major water bearing layers were encountered within the proposed depth of the spillway excavation.

No major difficulties with the spillway excavation are anticipated. It is expected that the shale and the weaker sandstones can be removed by ripping. Some blasting of cemented sandstone layers may be required. Occasional seepage zones may be encountered, however water quantities are expected to be low and water can be collected into drainage ditches and sumps using conventional construction practice.

Permanent cutslopes in the clay till should be no steeper than 3 horizontal to 1 vertical in order to ensure long term stability. Flatter sideslopes within the clay till may be desireable in order to liberate more material for use in the construction of the embankment.

Permanent cutslopes within the shale or sandstone bedrock could be trimmed at 1 horizontal to 1 vertical or steeper and they would remain stable. However, the exposed shale is susceptible to weathering and erosion which could become a major maintenance problem. For this reason, it is recommended that permanent slopes in the bedrock be cut no steeper than 2 horizontal to 1 vertical

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in order to minimize surface erosion. If steeper slopes are desired, surface treatment, such as shotcreting, should be considered.

It is understood that the spillway chute will be lined with concrete in order to prevent erosion. It is understood that the gradient for the spillway chute cannot exceed 3 horizontal to 1 vertical for hydraulic reasons. This slope will be satisfactory in the shale bedrock from a slope stability point of view. If necessary, steeper longitudinal slopes could be considered. In any event, it will be necessary to anchor the spillway chute to the slope and install suitable drainage below the chute to prevent uplift due to water pressure.

It is understood that the concrete weir and gate structure at the top of the spillway chute will be subjected to horizontal loads, which must be transferred to the underlying shale bedrock. The horizontal shearing resistance of the shale bedrock will be less than the shear strengths reported by PFRA because of the presence of bedding planes parallel to the direction of shear. For preliminary design of the weir, a nominal angle of friction of 20 degrees maybe used. This value should be confirmed during detailed design.

The shale bedrock can be expected to rebound due the weight of material removed from the excavation. Rebound will be primarily elastic and will therefore take place as the excavation proceeds. Past experience with deep excavations in materials similar to those encountered here, indicates that all rebound (including time dependent rebound) will be essentially complete 2 to 3 months after the excavation is completed. The total rebound will range from 1 to 3 inches. A more precise estimate of the rebound can be made once the configuration and depth of the excavation has been confirmed.

5.8 Low Level Conduit

In view of the highly variable compressibility of the alluvial sediments at this site, it is recommended that the low level conduit be placed near the north or south abutment, so that the conduit can be placed directly on bedrock. The drillhole information provided by PFRA suggests the north abutment is the more favourable location for the conduit.

It is recommended that cut-off collars be cast around the upstream portion of the conduit in Zone 1 in order to reduce the possibility of seepage. Equally important, the

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downstream portion of the conduit through Zone 2/3 should be surrounded by a minimum thickness of 3 feet of pervious filter in order to prevent possible erosion along the conduit.

Settlement and horizontal elongation of the conduit can be expected as the embankment fill is constructed. The conduit must be designed to accomodate these movements. The deformations will be minimized if the conduit is placed on the bedrock near one of the abutments as recommended.

5.9 Concrete

A suitable source of concrete aggregate has not been identified. The alluvial sands and gravels at the damsite are below the water table and it is unlikely they can be recovered and processed economically. Other potential sources for concrete aggregate should be investigated.

A number of water soluble sulphate tests were undertaken by PFRA and these indicate negligible water soluble sulphates are present in the soils at this site. These test results may not, however, be representative of the soils adjacent to concrete structures. Unless further water soluble sulphate tests are undertaken, it is recommended that Type 50 sulphate resistant cement be used for all concrete in contact with soil at this site. This is particularly important for the low level conduit, which is subject to an unfavourable hydraulic environment from the point of view of potential attack by sulphates.

5.10 Additional Gectechnical Investigations

If a decision is made to proceed with construction of a dam at Mosquito Creek Site 4, the following geotechnical work is recommended:

1) A hydrogeological investigation of the buried Silver Valley should be undertaken, including the drilling of test holes and insitu permeability tests, in order to provide a more reliable assessment of the potential for leakage from the reservoir into Silver Valley. This work should be undertaken before a final decision is made to proceed with construction of a dam at this site.

2) Test pits should be excavated in the alluvial deposits along the dam centreline and downstream from the dam. The purpose of this work is:

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- To identify the quality and quantity of granular materials suitable for use in embankment construction,

- to establish the depth to which the cutoff can be readily excavated above the water table,

- to establish the feasibility of installing a slurry trench wall, and

- to establish the depth to bedrock along the centreline of the proposed low level conduit, as well as to establish potential construction difficulties.

3) High pressure injection tests should be undertaken in the bedrock at the dam abutments in order to assess the stratigraphy and permeability of the bedrock at these locations.

4) Once the location and depth of the spillway has been confirmed, additional test holes should be drilled along the proposed centrelines. The purpose of these holes is:

   - To confirm subsurface conditions along the spillway centreline,

   - to confirm groundwater conditions along the spillway centreline, and

   - to sample and test the clay till in order to confirm the potential use of this material in embankment construction.

5) Additional test pits and test holes should be drilled in potential borrow areas for clay till. Specifically, it is necessary to determine the following:

   - The susceptibility of the clay till to internal piping (dispersion tests are recommended),

   - the water soluble sulphate content of the clay borrow, and

   - the compaction properties of the clay borrow.

6) It is recommended that pneumatic piezometers be installed at selected locations in the vicinity

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of the damsite in order to establish current piezometric pressures in the bedrock at the site. This information is required in order to evaluate the stability of the proposed embankment.

7) A suitable source of rip rap for upstream slope protection should be located.

8) A program of laboratory strength testing should be undertaken on samples of compacted silty clay till, on undisturbed alluvium and on undisturbed samples of bedrock core in order to establish the effective stress strength and pore pressure parameters of these materials for use in stability analyses.

9) An earth materials balance study should be carried out in order to optimize the configuration of the dam embankment as well as to establish the most efficient sequence of construction.

10) Selective stability analyses should be undertaken in order refine the design of the embankment.

11) Detailed field checking of the valley slopes along the reservoir should be undertaken in order to identify localized areas where the potential for instability is of concern.

12) A detailed hydrogeological assessment of the damsite and reservoir should be undertaken in order to assess the impact of the proposed reservoir on the groundwater regime. This work should include a more detailed evaluation of lateral seepage from the reservoir.

It is expected that 3 to 4 months would be required to complete the geotechnical investigations outlined above.

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SECTION 6

SQUAW COULEE SITE 2

6.1 General

Squaw Coulee Site 2 is located about 12 km south of High River, Alberta, as shown on Figure 1. The damsite is located in the South Half of Section 26, Township 17, Range 29, West of the 4th Meridian.

The proposed reservoir would have a full supply level at 1067 metres and would result in a reservoir about 9 km long. The dam embankment would have a maximum height of about 21 m, a crest length of about 370 m, and a freeboard of 3 m. The normal operating drawdown range is about 15 m.

A spillway would be constructed around the left abutment of the embankment. This spillway would require an excavation of 8 to 10 m below ground surface.

A low level conduit would be constructed through the embankment. The conduit would have an intake elevation at about 1050 m and would be about 1.8 m in diameter. Flows through the conduit would be controlled by gates accessed by a manhole immediately upstream of the centreline of the embankment.

6.2 Reservoir Geology

The surficial geology of the reservoir is shown in Appendix C, Drawing 6.1. Ground moraine, consisting of silty clay is the near surface soil on the uplands adjacent to this reach of Squaw Coulee. As indicated on Drawing 6.1, the valley bottom is infilled with recent alluvial deposits, consisting of silts and sands, and colluvium.

Squaw Coulee has been cut down through the glacial till and into the underlying bedrock. Bedrock is exposed along much of the valley walls; however, in most locations it is covered by a thin veneer of colluvium.

The valley walls are moderately steep, ranging up to 30 degrees. No evidence of deep seated slope instability is apparent in almost all locations on the valley walls within this reach of the stream. Shallow, localized instability of the colluvial materials occurs in a few locations. A possible deep seated landslide was observed about 3 km upstream from the proposed damsite.

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6.3 Reservoir Shoreline Stability

Impoundment of the reservoir will cause a decrease in the overall stability of the valley walls, mainly due to rapid drawdown and wave erosion. In addition, increased irrigation on the uplands adjacent to the reservoir will also tend to decrease the stability of the slopes. These effects may be significant, depending on local topography. However, the valley slopes should remain generally stable and deep seated large scale landslides are unlikely to occur along the major portion of the reservoir slopes.

The limits of any deep seated slope instability which does occur along the reservoir are expected to be confined within a zone described by a line drawn parallel to the valley at a distance of about 50 m back from the top of the valley walls. This set back could be reviewed for site specific cases.

The reservoir full supply level is located below the elevation of the bedrock surface along this reach of the coulee. Therefore, the reservoir shoreline will erode into thin colluvium which is underlain by shale and sandstone bedrock. It is expected that small scale, localized instability of the slopes will occur due to wave erosion along the shoreline, particularly in locations where uncemented sandstone is present at the shoreline. This instability will be confined to the range of drawdown in the reservoir. The instability is expected to be self stabilizing as beaches develop over a period of 20 to 30 years.

6.4 Reservoir Watertightness

Detailed information concerning the present ground water elevation adjacent to the proposed reservoir is not available. Information from small scale hydrogeological maps for this area indicates, however, that the ground water table elevation is about 1035 m or slightly higher. Assuming a reservoir elevation of 1067, it can be expected that the groundwater table in the vicinity of the reservoir will rise significantly.

Some lateral seepage into the valley walls will undoubtedly occur, primarily through joints and fractures in the bedrock, as well as through the more pervious clean sandstone beds. While the hydraulic gradient between the reservoir and the groundwater table is relatively high, the mass permeability of the bedrock is relatively low.

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Water leakage from the reservoir should not be excessive, however it can be expected to exceed the leakage which occurs at other damsites such as Little Bow Site 1. More detailed information is required in order to evaluate the probable magnitude of lateral seepage from the reservoir at Squaw Coulee Site 2.

No buried valleys have been reported along this portion of Squaw Coulee or in the vicinity of the damsite. Leakage into pervious zones within such valleys is therefore not identified as a concern at this time.

6.5 Damsite Geology

Investigations were not carried out previously by PFRA at Squaw Coulee Site 2. Three test holes were drilled during this recent study in order to obtain preliminary information on subsurface conditions at the site. Due to wet surface conditions, it was not possible to access the area in the immediate vicinity of the proposed damsite. The three test holes were drilled further downstream at the locations shown on Drawing 6.2. Examination of airphotos indicates that the subsurface conditions encountered at the test hole locations should be fairly representative of any location along this portion of Squaw Coulee, including Site 2. The test hole logs are presented in Appendix C.

6.5.1 Surface Conditions

The surficial deposits, and other pertinent features of Squaw Coulee Site 2 are presented in Appendix C, Drawing 6.2.

The near surface soils on the uplands in the vicinity of the damsite are morainal deposits consisting of silty clay.

The valley slopes near the damsite are covered by a thin veneer of colluvial material over bedrock. The valley slopes are moderate, ranging from about 10° (about 6:1) at the east abutment to about 13° (about 4.5:1) at the west abutment. No evidence of deep seated instability of the valley walls is apparent in the vicinity of the damsite.

The valley bottom is U-shaped and has a mantel of recent alluvial and colluvial deposits overlying bedrock. The stream in Squaw Coulee is an underfit stream occupying an early post-glacial spillway or meltwater channel.

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6.5.2 Subsurface Conditions

Subsurface conditions at the damsite are presented in Appendix C, Drawing 6.3. This drawing has been prepared by extrapolating subsurface conditions, as encountered in the test holes, to Squaw Coulee Site 2. The following major soil units are expected at Site 2:

- clay till
- alluvium
- bedrock

Clay Till

Silty clay till is expected to overlie the bedrock on the uplands on both sides of the damsite. The clay till was found to range up to 8 m in thickness in the test holes. Cuts along the secondary highway immediately south of Site 2, expose 4 to 5 m of till over shale bedrock. Till thicknesses can be expected to increase at locations further from the top of the valley slope. The clay till is very stiff to hard in consistency and has a low to medium plasticity. The clay till is expected to have properties similar to those determined by PFRA at Mosquito Creek Site 4 (Table 5.1).

Alluvium

Alluvial materials, and some colluvium, can be expected to be encountered in the valley bottom. These materials will be highly variable in texture, ranging from silty clays to sands and gravels. A sandy silt was encountered in test hole 85-4, which was drilled further downstream in Squaw Coulee. The alluvium at that location was found to be about 5 m thick.

It is expected that cohesive alluvial deposits will range from firm to very stiff in consistency, while the cohesionless alluvial deposits will be loose to compact.

Bedrock

Bedrock consists of interbedded shales and sandstones. The bedrock can be expected to cover the

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entire spectrum from shale, sandy shale, silty shale, shaly sandstone to sandstone. The various lithologies will blend gradually from one to another. There appears to be a significant greater proportion of cemented sandstone at Squaw Coulee Site 2, than is present at either of the other two damsites.

The properties of the shale at Squaw Coulee Site 2 are expected to be similar to those reported by PFRA for Mosquito Creek Site 4 (Table 5.1). The shale would be classified as a weak rock. The shale is expected to weather rapidly to a medium to high plastic clay when left exposed at ground surface.

No slickensides or other planes of weakness were noted in any of the core taken from the recent drill holes. Most significantly, no deep seated bedrock failures have been observed along the valley slopes immediately upstream and downstream of the proposed damsite. Such instability is common in locations where pre-existing failure surfaces are found in the Cretaceous sediments of Western Canada. On the basis of the foregoing information, it has been concluded that peak strength parameters (rather than much lower residual strength parameters) will be acceptable for use in design of the dam embankment and other structures at this location.

The properties of the cemented sandstone at this site will be similar to those summarized in Table 5.1 for Mosquito Creek Site 4. The sandstone can be classified as a moderately strong rock. These stronger sandstones are clay and calcite cemented. The cemented sandstones are expected to break down when exposed to water for prolonged periods. The distribution of the stronger cemented sandstones within the bedrock at this site is not known.

Groundwater

No seepage zones or free water was encountered in any of the test holes drilled on the uplands near Squaw Coulee. Water levels in the alluvial deposits on the valley floor are assumed to be at about the same elevation as water levels in the small stream at the bottom of the coulee.

Pneumatic piezometers were not installed in the test holes and hence piezometric elevations within

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the impervious bedrock formations at this site are not known. A detailed hydrogeological assessment of the site has not been undertaken.

6.6 Dam Embankment

6.6.1 Siting

There are no significant geotechnical factors affecting the positioning of the embankment centreline at this site. Subsurface conditions are not expected to vary significantly for that portion of the Coulee from about 0.5 km upstream from Site 2 to about 1 km downstream of Site 2.

6.6.2 Embankment Configuration

A tentative configuration for the proposed dam embankment is presented in Appendix C, Drawing 6.4. It is expected that the configuration presented on Drawing 6.4 would be refined and optimized during detailed design, as a result of material balance studies and detailed slope stability analyses.

An average upstream slope of 4 horizontal to 1 vertical is recommended, with an average downstream slope of 3 horizontal to 1 vertical. Both the upstream and downstream slopes may be bermed in order to reduce the volume of material required. A decision in this regard would be made during the detailed design phase.

The upstream portion of the dam (Zone 1) would be constructed of medium plastic silty clay till. This material would have to come from borrow sources located on the uplands adjacent to the damsite. No silty clay till will be liberated from the spillway excavations. The silty clay till is 2 or 3% dry of optimum water content and it will be necessary to add water during compaction.

The downstream portion of the dam (Zone 2/3) would be constructed of random semipervious to pervious silts and sands taken initially from the cutoff excavation. Additional material for this portion of the dam would have to come from borrow sources located on the valley floor, upstream from the dam. The coarser and more pervious materials should be placed towards the downstream face of the dam.

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Compacted shale and sandstone bedrock from the spillway excavations could also be used in the downstream portion of the dam as indicated on the drawing. As noted, the exact boundaries of the various zones can be modified to make optimum use of the available materials.

Clean sand and gravel would be placed to form the toe drain, as shown on the drawing. It is unlikely that suitable sand or gravel is present in the valley and an alternative source for this material has not been identified. Filter cloth could be used, as indicated on the drawing, to prevent migration of fines into the toe drain.

Preliminary calculations indicate that the dam foundation can be expected to settle in the order of 1 foot or so during construction. The dam crest would settle an additional 6 inches or so when the reservoir is filled. It may be desirable to raise the finished dam crest above the design elevation in order to accommodate movements of this order. If required, a detailed settlement analysis of the embankment can be undertaken during detailed design.

6.6.3 Seepage Control

All organic material and all soft or loose inorganic materials should be stripped from below the area of the embankments. The entire area should be scarified and recompacted to a depth of at least 1 foot. Proof rolling is recommended prior to placing fill in order to identify any soft zones. Waste soil should be placed upstream of the dam preferrably in the existing creek channel.

Pervious layers within the alluvium represent the most serious source of seepage below the embankment. A positive cutoff, extending into the shale bedrock below the embankment is recommended. The width of the base of the cutoff could vary from a minimum 30 feet to 50 feet (or greater). Seepage into the cutoff excavation and the stability of the slopes of the cutoff excavation are a concern. It is recommended that test pits be excavated along the centreline of the cutoff in order to provide a better assessment of the behaviour of excavation sideslopes.

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Seepage through fractures and joints in the bedrock below the dam is a concern. It may be necessary to extend the toe drain down into the bedrock in order to relieve potential uplift pressures at that point. Alternatively, pressure relief wells should also be considered. The requirement in this regard will depend on the bedrock stratigraphy below the dam centreline and below the downstream toe.

Abutment seepage is a major concern. The precise requirements to control this source of seepage cannot be made until data from additional field permeability tests in the bedrock is obtained. It may be necessary to inject grout into the abutments in order to control seepage. The bedrock at the abutments of the dam would likely be stepped, in order to facilitate the placement and compaction of fill against the abutments. Seepage along the interface between the compacted clay fill and the bedrock may require shotcreting of the bedrock surface in that area where the impervious clay core is placed against the abutment. Silty clay compacted against the abutments should be compacted at 2 or 3% wet of optimum. Further, a clean gravel layer should be placed against the bedrock surface immediately downstream from the impervious zone. This gravel layer should be connected to a pipe and, if practical, discharged to a location where it can be monitored. This gravel layer will drain any seepage water from the bedrock and the bedrock contact.

6.6.4 Upstream Slope Protection

The upstream face of the dam must be armoured within the normal operating reservoir levels in order to reduce possible wave erosion. As indicated on Drawing 6.4, the slope protection should consist of 0.5 m of well graded sand and gravel covered by 0.6 m of rock armour. The class of rock armour required will be established during detailed design.

A suitable source of rip rap slope protection has not been established. The local cemented sandstone is unsuitable for rip rap because it breaks down when exposed to water for prolonged periods.

An often acceptable alternative to expensive rip rap is to use coarse pit run gravel and accept the need for occasional regrading of the slopes. The use of soil cement could also be considered.

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6.6.5 Downstream Slope Protection

Measures will be required to prevent erosion of the downstream face of the embankment due to surface runoff. The crest of the dam should be surfaced with gravel (for trafficability) and sloped towards the reservoir. The downstream face of the embankment should be covered with 1 foot of topsoil and hydro-seeded.

6.7 Spillway

Construction of the spillway approach channel around the left abutment will require an excavation to a depth of 6 to 10 m below ground surface. The excavation will be made through about 1 metre of colluvium and into the underlying shale and sandstone bedrock. No major water bearing layers are expected within the proposed depth of excavation.

No major difficulties with the spillway excavation are anticipated. It is expected that the shale and the weaker sandstones can be removed by ripping. Some blasting of cemented sandstone layers may be required. Occasional seepage zones may be encountered, however water quantities are expected to be low and water can be collected into drainage ditches and sumps using conventional construction practice.

Permanent cutslopes in the clay till or colluvium overlying the bedrock should be no steeper than 3 horizontal to 1 vertical in order to ensure long term stability.

Permanent cutslopes within the shale or sandstone bedrock could be trimmed at 1 horizontal to 1 vertical or steeper and they would remain stable. However, the exposed shales are expected to be susceptible to weathering and erosion which could become a major maintenance problem. For this reason, it is recommended that permanent slopes in the bedrock be cut no steeper than 2 horizontal to 1 vertical in order to minimize surface erosion. If steeper slopes are desired, surface treatment, such as shotcreting, should be considered.

It is understood that the spillway chute will be lined with concrete in order to prevent erosion. It is understood that the gradient for the spillway chute cannot exceed 3 horizontal to 1 vertical for hydraulic reasons. This slope will be satisfactory in the shale bedrock from a slope stability point of view. If necessary, steeper

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longitudinal slopes could be considered. In any event it will be necessary to anchor the spillway chute to the slope and install suitable drainage below the chute to prevent uplift due to water pressure.

It is understood that the concrete weir and gate structure at the top of the spillway chute will be subjected to horizontal loads, which must be transferred to the underlying shale bedrock. For preliminary design of the weir, a nominal angle of friction of 20 degrees may be used. This value should be confirmed during detailed design.

The shale bedrock can be expected to rebound due to the weight of material removed from the excavation. Rebound will be primarily elastic and will therefore take place as the excavation proceeds. Past experience with deep excavations in materials similar to those encountered here, indicates that all rebound (including time dependent rebound) will be essentially complete 2 to 3 months after the excavation is completed. The total rebound will range from 1 to 3 inches. A more precise estimate of the rebound can be made, if necessary, once the configuration and depth of excavation has been confirmed.

6.8 Low Level Conduit

In view of the highly variable compressibility of the alluvial sediments at this site, it is recommended that all alluvial materials below the low level conduit through Zone 1 be subexcavated down to bedrock, and replaced with well compacted silty clay. It is therefore desireable to place the conduit in a location where the depth to bedrock is at a minimum. Test holes will have to be drilled along the embankment centreline in order to establish the most suitable location for the conduit.

It is recommended that cutoff collars be cast around the upstream portion of the conduit in Zone 1 in order to reduce the possibility of seepage. Equally important, the downstream portion of the conduit through Zone 2/3 should be surrounded by a minimum thickness of 1 metre of pervious filter in order to prevent possible erosion along the conduit.

Settlement and horizontal elongation of the conduit can be expected as the embankment fill is constructed. The conduit must be designed to accomodate these movements. The deformations could be minimized if the conduit were placed directly on the bedrock.

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6.9 Concrete

A suitable source of concrete aggregate has not been identified. It is anticipated that suitable aggregate will not be available in the alluvial deposits on the valley floor in the immediate vicinity of the damsite.

Water soluble sulphate tests were not undertaken on samples of clay till, because none of the test holes drilled are located in potential borrow sources. Unless further water soluble sulphate tests are undertaken, it is recommended that Type 50 sulphate resistant cement be used for all concrete in contact with soil at this site. This is particularly important for the low level conduit, which is subject to an unfavourable hydraulic environment from the point of view of potential attack by sulphates.

6.10 Additional Geotechnical Investigations

If a decision is made to proceed with construction of a dam at Squaw Coulee Site 2, an extensive program of test drilling should be undertaken at the proposed damsite. The investigation should be similar in scope to that undertaken by PFRA at Little Bow Site 1. In addition the following work should be undertaken:

1) Test pits should be excavated in the alluvial deposits along the dam centreline and upstream of the dam. The objectives of this work are:

   - To identify the quality and quantity of granular materials for use in embankment construction,

   - to establish the depth to bedrock along the centreline of the proposed cutoff, as well as to identify potential difficulties in making the excavation for the cutoff,

   - to establish the depth to bedrock along the centreline of the proposed low level conduit, as well as to establish potential construction difficulties, and

   - to establish the depth to bedrock along the centreline of the proposed toe drain.

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2) High pressure injection tests should be undertaken in the bedrock, at the abutments, along the dam centreline and near the toe of the embankment, in order to assess the stratigraphy and permeability of the bedrock at these locations.

3) Once the location and depth of the spillway has been confirmed, test holes should be drilled along the proposed centreline. The purpose of these holes is:

   - To confirm subsurface conditions along the spillway centreline and in particular to establish the volume of cemented sandstone which must be blasted,

   - to confirm groundwater conditions along the spillway centreline, and

   - to sample and test the shale bedrock in order to assess the potential use of this material in embankment construction.

4) Additional test pits and test holes should be drilled in potential borrow areas for clay till. Specifically, it is necessary to determine the following:

   - The susceptibility of the clay till to internal piping (dispersion tests are recommended),

   - the water soluble sulphate content of the clay borrow, and

   - the compaction properties of the clay borrow.

5) Potential sources for concrete aggregate should be identified. Test pits should be excavated in each source in order to establish the quantity and quality of the aggregates.

6) It is recommended that pneumatic piezometers be installed at selected locations in the vicinity of the damsite in order to establish current piezometric pressures in the bedrock at the site. This information is required in order to evaluate the stability of the proposed embankment.

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7) A program of laboratory strength testing should be undertaken on samples of compacted silty clay till, on undisturbed alluvium and on undisturbed samples of bedrock core in order to establish the effective stress strength and pore pressure parameters of these materials for use in stability analyses.

8) A suitable source of rip rap for upstream slope protection should be located.

9) An earth materials balance study should be carried out in order to optimize the configuration of the dam embankment as well as to establish the most efficient sequence of construction.

10) Selective stability analyses should be undertaken in order to refine the design of the embankment.

11) Detailed field checking of the valley slopes along the reservoir should be undertaken in order to identify localized areas where the potential for instability is of concern.

12) A detailed hydrogeological assessment of the damsite and reservoir should be undertaken in order to assess the impact of the proposed reservoir on the groundwater regime. This work should include a more detailed evaluation of lateral seepage from the reservoir.

It is expected that 4 to 6 months would be required to complete the geotechnical investigations outlined above.

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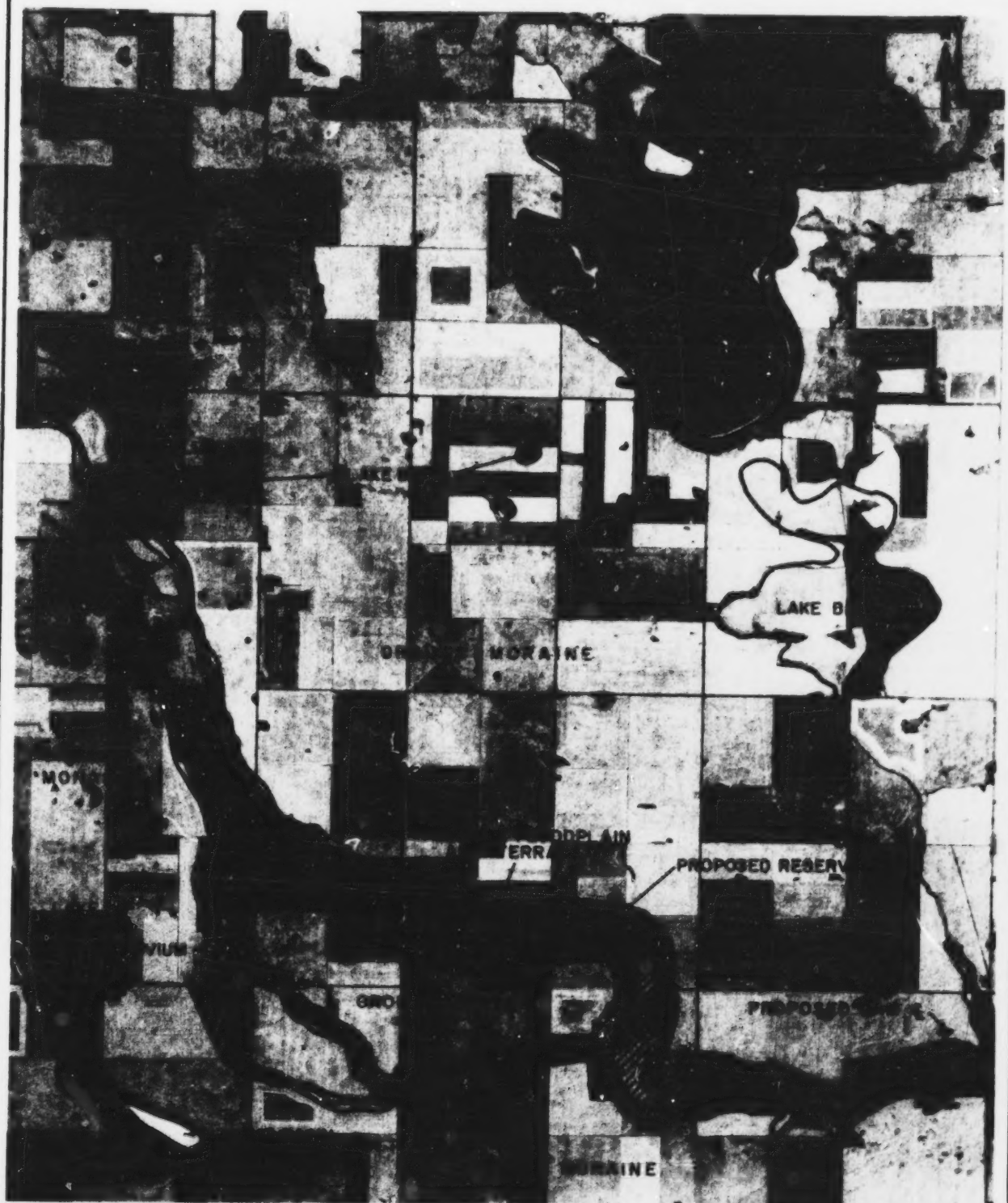
APPENDIX A
TO GEOTECHNICAL REPORT

LITTLE BOW SITE 1

Drawing 4.1 Surficial Geology - Reservoir
Drawing 4.2 Surficial Geology - Damsite
Drawing 4.3 Geological Cross-Section
Drawing 4.4 Preliminary Embankment Design

Test Hole Log 85-1



////// INDICATES APPROXIMATE LIMITS OF RESERVOIR

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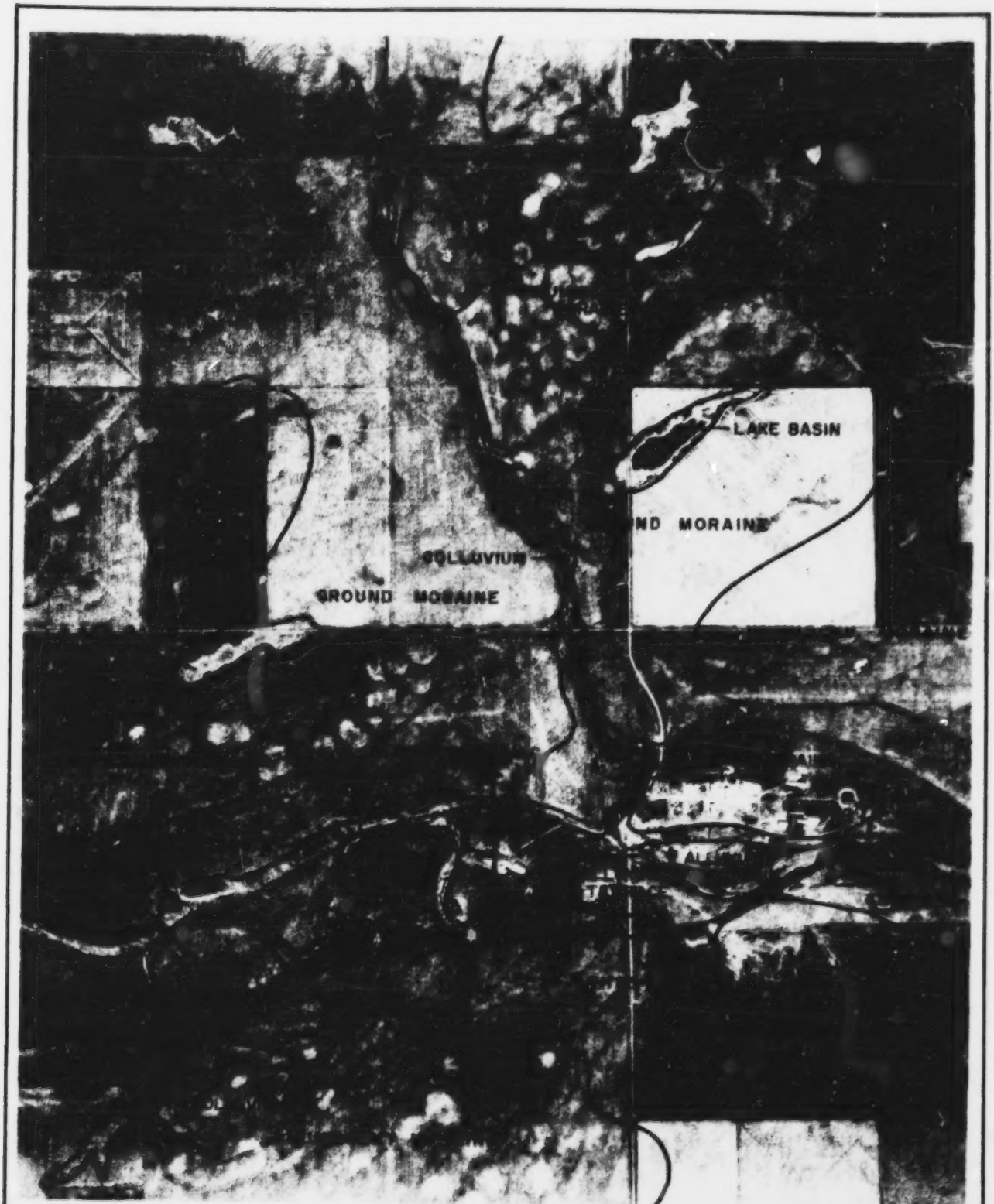

W-E-R ENGINEERING LTD.

SURFICIAL GEOLOGY
LITTLE BOW SITE 1
DAMSITE

THURBER CONSULTANTS LTD., Geotechnical Engineers

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DATE	MAY 1985
APPROVED	
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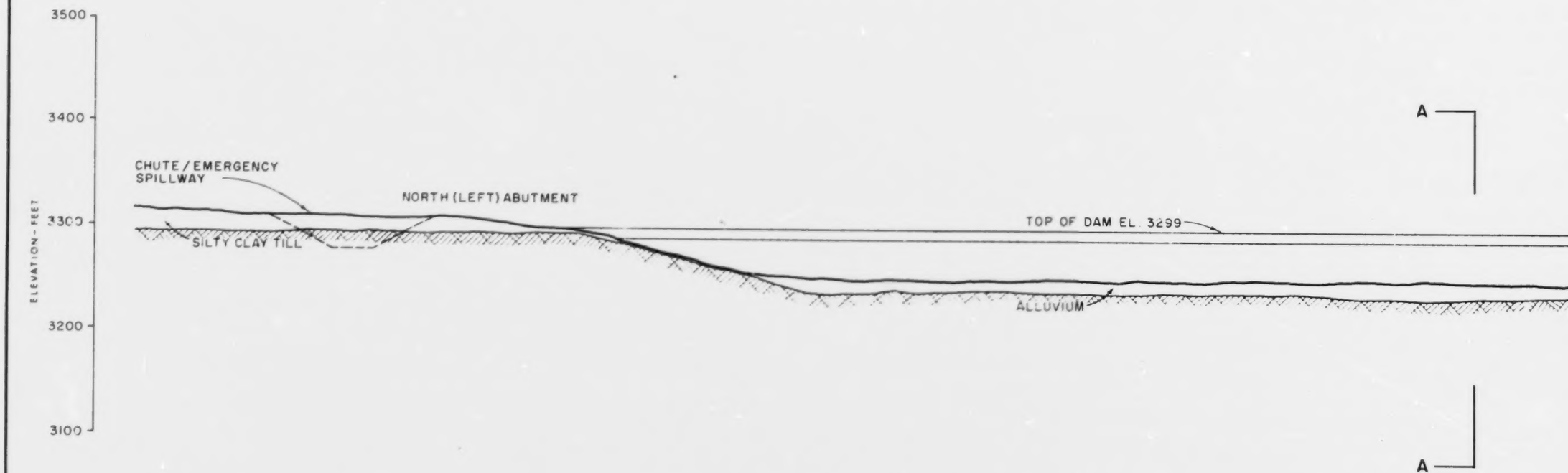
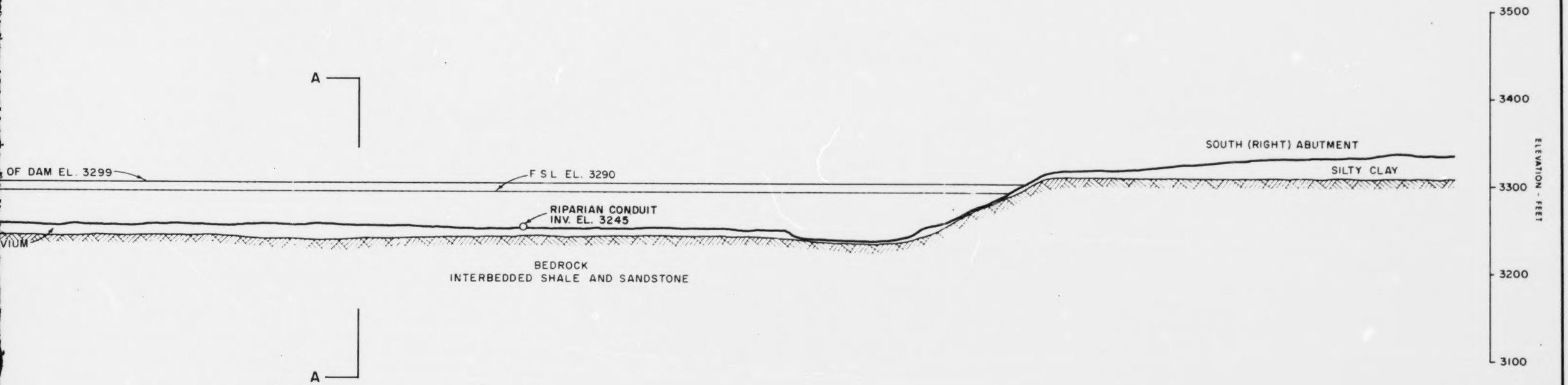


NOTES
1. CROSS-SECTION PREPARED FROM INF
IN PFRA REPORT ENTITLED "PRELIM
REPORT, LITTLE BOW RIVER PROJEC
FEBRUARY 14, 1968.
2. REFER TO DRAWING 4.4 FOR CROSS-



NOTES
1. CROSS-SECTION PREPARED FROM INFORMATION PRESENTED IN PFRA REPORT ENTITLED "PRELIMINARY SOIL MECHANICS REPORT, LITTLE BOW RIVER PROJECT - SITE 1" DATED FEBRUARY 14, 1968.
2. REFER TO DRAWING 4.4 FOR CROSS-SECTION A-A

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GEOLOGICAL CROSS-SECTION
LITTLE BOW SITE 1

THURBER CONSULTANTS LTD., Geotechnical Engineers

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FILE NO.	19-293-0
DATE	MAY 1985
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SCALE	1" = 100'
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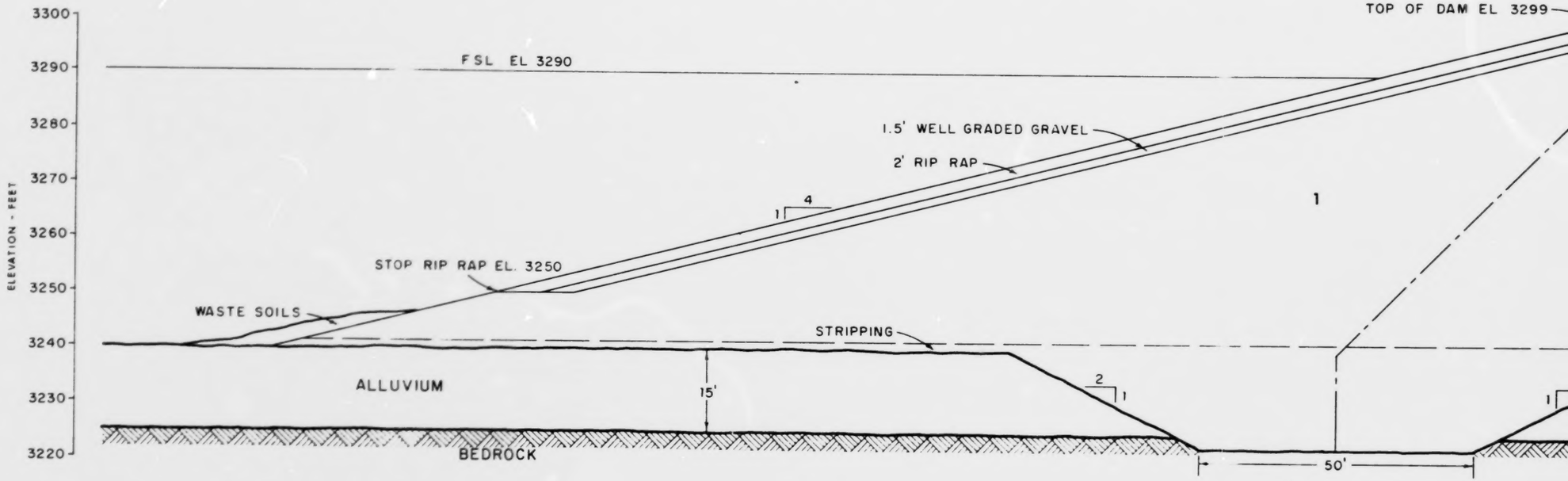
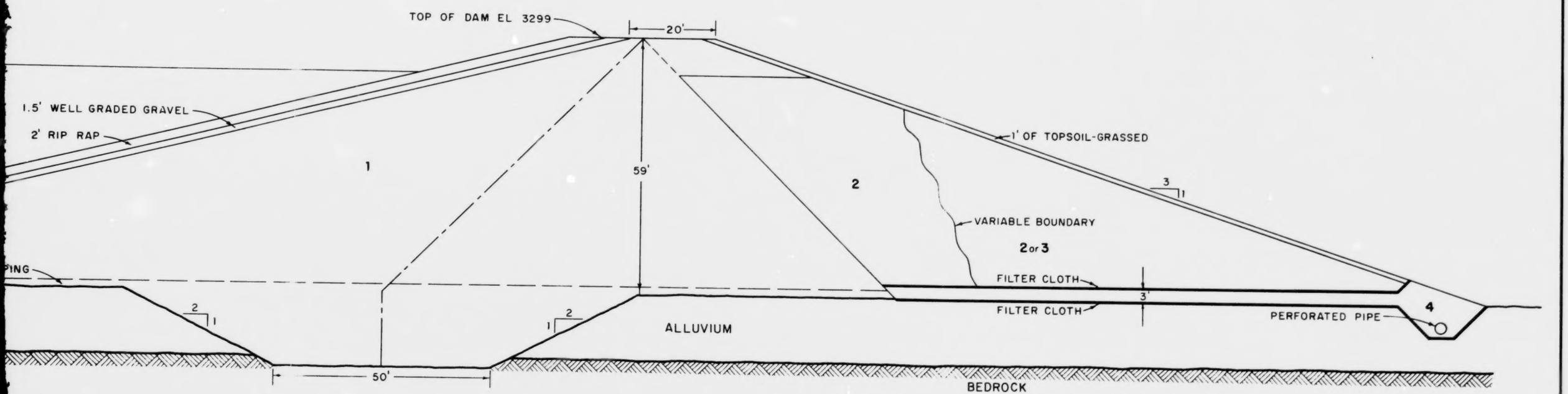


1 IMPERVIOUS MATERIAL. WELL COMPACTED SILTY CLAY TILL FROM UPLANDS.

2 PERVIOUS TO SEMI-PERVIOUS MATERIAL. RANDOM SANDS AND GRAVELS FROM CUT OFF EXCAVATION OR FROM BORROW AREAS IN ALLUVIUM.

3 PERVIOUS TO SEMI-PERVIOUS MATERIAL. RECOMPACTED SHALE OR SANDSTONE BEDROCK FROM SPILLWAY EXCAVATION.

4 PERVIOUS DRAIN. CLEAN SAND AND GRAVEL WITH LESS THAN 3% PASSING THE NO. 8 SIEVE.



PERVIOUS MATERIAL. RECOMPACTED
E BEDROCK FROM SPILLWAY EXCAVATION.

EAN SAND AND GRAVEL WITH LESS
THE NO. 8 SIEVE.

W-E-R ENGINEERING LTD.

PRELIMINARY EMBANKMENT DESIGN
LITTLE BOW SITE 1

THURBER CONSULTANTS LTD., Geotechnical Engineers

DRAWN	LBS/JAB
FILE NO.	19-293-0
DATE	MAY 1985
APPROVED	
SCALE	1" = 20'
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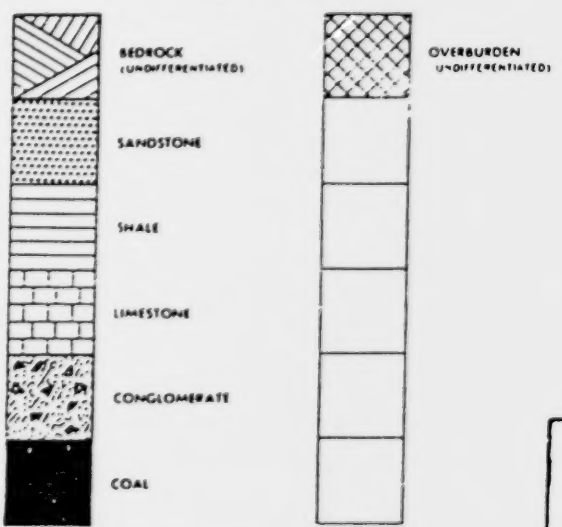
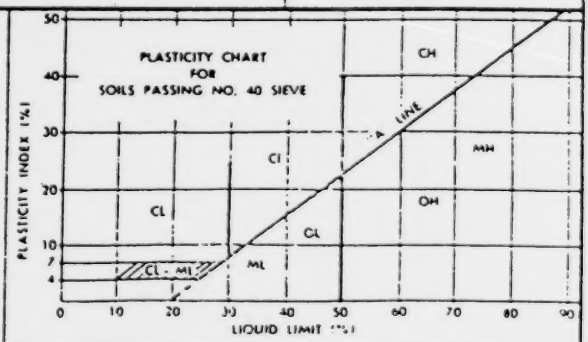
MODIFIED UNIFIED CLASSIFICATION SYSTEM FOR SOILS

MAJOR DIVISION			GROUP SYMBOL	GRAPH SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA	
COARSE-GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN 200 SIEVE)	GRAVELS (MORE THAN HALF COARSE FRACTION LARGER THAN NO 4 SIEVE)	CLEAN GRAVELS (LITTLE OR NO FINES)	GW		WELL GRADED GRAVELS, LITTLE OR NO FINES	$C_U = \frac{D_{60}}{D_{10}} > 4 \quad C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
			GP		POORLY GRADED GRAVELS, AND GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		DIRTY GRAVELS (WITH SOME FINES)	GM		SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW "A" LINE OR P.I. LESS THAN 4
			GC		CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE "A" LINE P.I. MORE THAN 7
	SANDS (MORE THAN HALF FINE GRAINS SMALLER THAN NO 4 SIEVE)	CLEAN SANDS (LITTLE OR NO FINES)	SW		WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$C_U = \frac{D_{60}}{D_{10}} > 6 \quad C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
			SP		POORLY GRADED SANDS, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		DIRTY SANDS (WITH SOME FINES)	SM		SILTY SANDS, SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW "A" LINE P.I. LESS THAN 4
			SC		CLAYEY SANDS, SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE "A" LINE P.I. MORE THAN 7
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT PASSES 200 SIEVE)	SILTS (BELOW "A" LINE NEGLIGIBLE ORGANIC CONTENT)	$W_L < 50\%$	ML		INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (see below)	
		$W_L > 50\%$	MH		INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS		
	CLAYS (ABOVE "A" LINE ON PLASTICITY CHART NEGLIGIBLE ORGANIC CONTENT)	$W_L < 30\%$	CL		INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAYS, LEAN CLAYS		
		$30\% < W_L < 50\%$	CI		INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS		
		$W_L > 50\%$	CH		INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS		
	ORGANIC SILTS & CLAYS (BELOW "A" LINE OR CHART)	$W_L < 50\%$	OL		ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	WHENEVER THE NATURE OF THE FINE CONTENT HAS NOT BEEN DETERMINED, IT IS DESIGNATED BY THE LETTER "F", E.G SF IS A MIXTURE OF SAND WITH SILT OR CLAY	
		$W_L > 50\%$	OH		ORGANIC CLAYS OF HIGH PLASTICITY		
HIGHLY ORGANIC SOILS			Pt		PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOR OR ODOR, AND OFTEN FIBROUS TEXTURE	

SPECIAL SYMBOLS





1. ALL SIEVE SIZES MENTIONED ON THIS CHART ARE U.S. STANDARD, A.S.T.M. E.11.
2. BOUNDARY CLASSIFICATIONS POSSESSING CHARACTERISTICS OF TWO GROUPS ARE GIVEN COMBINED GROUP SYMBOLS, E.G. GW-GC IS A WELL GRADED GRAVEL SAND MIXTURE WITH CLAY BINDER BETWEEN 5% AND 12%.

THURBER CONSULTANTS LTD.
Geotechnical Engineers
EDMONTON VANCOUVER VICTORIA

SYMBOLS AND TERMS USED ON TEST HOLE LOGS

1. VISUAL TEXTURAL CLASSIFICATION OF MINERAL SOILS

CLASSIFICATION	APPARENT PARTICLE SIZE
Boulders	Greater than 200mm
Cobbles	75mm to 200mm
Gravel	5mm to 75mm
Sand	Not visible to 5mm
Silt	Non-Plastic particles, not visible to the naked eye
Clay	Plastic particles, not visible to the naked eye

2. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	APPROXIMATE UNDRAINED SHEAR STRENGTH	
Very Soft	Less than 10kPa	
Soft	10 - 25kPa	
Firm	25 - 50kPa	
Stiff	50 - 100kPa	
Very Stiff	100 - 200kPa	Modified from National Building Code
Hard	200 - 300kPa	
Very Hard	Greater than 300kPa	

3. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	STANDARD PENETRATION TEST (Number of Blows per 300mm)	
Very Loose	0 - 4	
Loose	4 - 10	
Compact	10 - 30	
Dense	30 - 50	Modified from National Building Code
Very Dense	Over 50	

4. LEGEND FOR TEST HOLE LOGS

- ○ Water content (% by weight) as determined on disturbed samples.
- ● Water content (% by weight) as determined on undisturbed samples.
- Disturbed bag or split spoon sample.
- Undisturbed Shelby Tube sample or core from VTM core barrel.
- No recovery
- Number of blows per 300mm for STANDARD PENETRATION TEST.
- C_u Undrained Shear Strength determined by unconfined compression test.
- C_{vane} Shear Strength determined by pocket vane.
- C_{pen} Shear Strength determined by pocket penetrometer.
- ▼ Water Level.

THURBER CONSULTANTS LTD. Sheet 1 of 5
Geotechnical Engineers

TEST HOLE Nº 85-1

LOG OF TEST HOLE

LOCATION South abutment, 120 m east of centre line

CLIENT W.E.R. Engineering Ltd.

PROJECT Little Bow River Basin Study

DATE March 13, 1985

METHOD Auger/coring

DRILLING Cº Mobile Augers Ltd.

INSPECTOR C. Minogue

LEGEND:
SAMPLES: Disturbed; Undisturbed; No recovery
WATER CONTENT:
WATER LEVEL; W_L LIQUID LIMIT; W_p PLASTIC LIMIT

WATER CONTENT (%) 10 20 30 40 50	ELEVATION (METRES)	REMARKS	UNIFIED SOIL CLASSIFICATION SYSTEM	LOG	DEPTH (METRES)
	TOP OF HOLE ELEV			TOPSOIL silt and sand, organic, brown	0
				SILT sandy, light brown, dry to damp	
				CLAY (TILL) silty, some sand, trace gravel, stiff to very stiff, low to medium plastic, brown, damp	
o		N=8/15/26	CL-CI		1
					2
				--traces of gypsum crystals and oxide specks	
o			CI		3
				--firm to stiff, moist, grey-brown	
o		N=6/8/12	CI-CH		4
					5

SCALE: 1:50 FILE Nº 19-293-0

THURBER CONSULTANTS LTD.
Geotechnical Engineers

Sheet 2 of 5

TEST HOLE No 85-1

LOG OF TEST HOLE

LOCATION South abutment, 120 m east of centre line

CLIENT W.E.R. Engineering Ltd.
PROJECT Little Bow River Basin Study
DATE March 13, 1985
METHOD Auger/Coring
DRILLING Co Mobile Augers Ltd.
INSPECTOR G. Minaruk

LEGEND:
SAMPLES: Disturbed, Undisturbed, No recovery
WATER CONTENT:
WATER LEVEL
W_L LIQUID LIMIT
W_P PLASTIC LIMIT

WATER CONTENT (%) 10 20 30 40 50	ELEVATION (METRES)	REMARKS	UNIFIED SOIL CLASSIFICATION SYSTEM	LOG	DEPTH (METRES)
	TOP OF HOLE ELEV.				5
o				--stiff	
					6
		N=6/9/19	CL-CI		7
o				--brown-dark grey	
					8
o			CH	--very stiff, dark grey, high plastic	
					9
				--inclusion of olive brown mudstone	
					10

SCALE: 1:50

FILE No 19-293-6

THURBER CONSULTANTS LTD.
Geotechnical Engineers

Sheet 3 of 5 | TEST HOLE № 85-1

LOG OF TEST HOLE

LOCATION South abutment
120 m east of centre line

CLIENT W.E.R. Engineering Ltd.
PROJECT Little Bow River Basin Study
DATE March 13, 1985
METHOD Auger/Coring
DRILLING Co Mobile Augers Ltd.
INSPECTOR C. Minogue

LEGEND:
SAMPLES: Disturbed; Undisturbed; No recovery
WATER CONTENT:
WATER LEVEL
w_L LIQUID LIMIT
w_P PLASTIC LIMIT

WATER CONTENT (%) 10 20 30 40 50

ELEVATION (METRES) — TOP OF HOLE ELEV.

REMARKS

LOG

DEPTH (METRES)

REMARKS	LOG	DEPTH (METRES)
N=61 for 15 cm Coring commenced at 10.7 m. No groundwater encountered prior to coring.	CLAY (TILL) continued	10
	SILTSTONE hard, grey	
	MUDSTONE green-grey, nuggetty	11
@ 11.11 m; sub-horizontal, clay-infilled bedding plane fracture Run 1: R=82% R.Q.D.=53% @ 11.81 m; bedding plane fracture, slickensided	SILTSTONE muddy, green-grey Sandstone lens, fine grained, grey	12
Run 2: R=95% R.Q.D.=77%	MUDSTONE silty, green-grey, locally nuggetty carbonaceous, grey	13
	SILTSTONE some fine sand, green-grey poorly cemented, some fissility	14
Run 3: R=96% R.Q.D.=36%	MUDSTONE with SILTSTONE lenses mudstone: silty, nuggetty, green-grey siltstone: poorly cemented	15

UNIFIED SOIL CLASSIFICATION SYSTEM

SCALE: 1:50 FILE № 19-293-0

THURBER CONSULTANTS LTD.
Geotechnical Engineers

Sheet 4 of 5

TEST HOLE № 85-1

LOG OF TEST HOLE

LOCATION South abutment, 120 m east of centre line

CLIENT W.E.R. Engineering Ltd.

PROJECT Little Bow River Basin Study

DATE March 13, 1985

METHOD Auger/Coring

DRILLING Co Mobile Augers Ltd.

INSPECTOR C. Minogue

LEGEND:
SAMPLES: Disturbed; Undisturbed; No recovery
WATER CONTENT:
WATER LEVEL
W_L LIQUID LIMIT
W_P PLASTIC LIMIT

WATER CONTENT (%) 10 20 30 40 50

ELEVATION (METRES) — TOP OF HOLE ELEV.

REMARKS

LOG

DEPTH (METRES)

Run 4: R=81%
R.Q.D.=68%

Run 5: R=81%
R.Q.D.=19%

@17.37 m: subvertical, joint, planar smooth, weathered

Run 6: R=78%
R.Q.D.=34%

@19.45-19.55m: vertical fracture, planar, rough

UNIFIED SOIL CLASSIFICATION SYSTEM

15 — MUDSTONE with SILTSTONE (continued)

16 — SILTSTONE
Muddy
green-grey

17 — SANDSTONE
fine grained, grey, organic-rich laminations, some poorly (clay) cemented zones
fine to medium, well-cemented

18 — MUDSTONE with SILTSTONE and SANDSTONE lenses
- mudstone: silty to very silty, locally carbonaceous
- siltstone: some fine sand, organic laminations
- sandstone: fine grained with organic laminations

19

20

SCALE: 1:50

FILE № 19-293-8

THURBER CONSULTANTS LTD.
Geotechnical Engineers

Sheet 5 of 5

TEST HOLE № 85-1

LOG OF TEST HOLE

LOCATION South abutment, 120 m east of centre line

CLIENT W.E.R. Engineering Ltd.
PROJECT Little Bow River Basin Study
DATE March 13, 1985
METHOD Augur/Coring
DRILLING CO Mobile Augers Ltd.
INSPECTOR C. Minogue

LEGEND:
SAMPLES: Disturbed; Undisturbed; No recovery
WATER CONTENT:
WATER LEVEL
w_L LIQUID LIMIT
w_p PLASTIC LIMIT

WATER CONTENT (%) 10 20 30 40 50	ELEVATION (METRES)	REMARKS	LOG	DEPTH (METRES)
	TOP OF HOLE ELEV.	Run 7: R=98% R.Q.D.=36%	MUDSTONE with SILTSTONE lenses (continued)	20
				21
		Run 8: R=74% R.Q.D.=33%	SANDSTONE fine to very fine, medium grey	22
			End of hole at 22.25 m Hole grouted in at completion	23

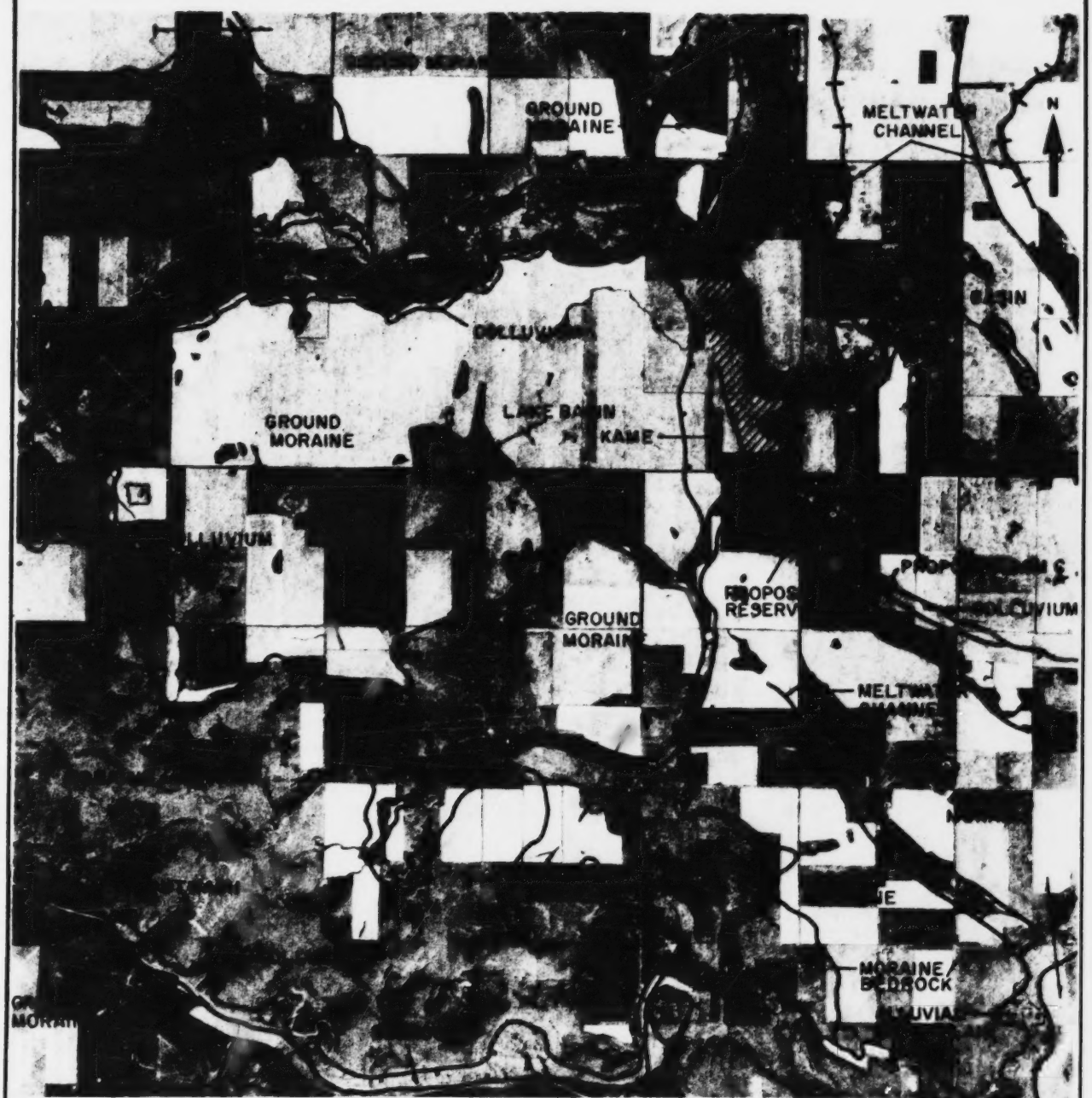
UNIFIED SOIL CLASSIFICATION SYSTEM

SCALE: 1:50 FILE № 19-293-0

APPENDIX B
TO GEOTECHNICAL REPORT

MOSQUITO CREEK SITE 4

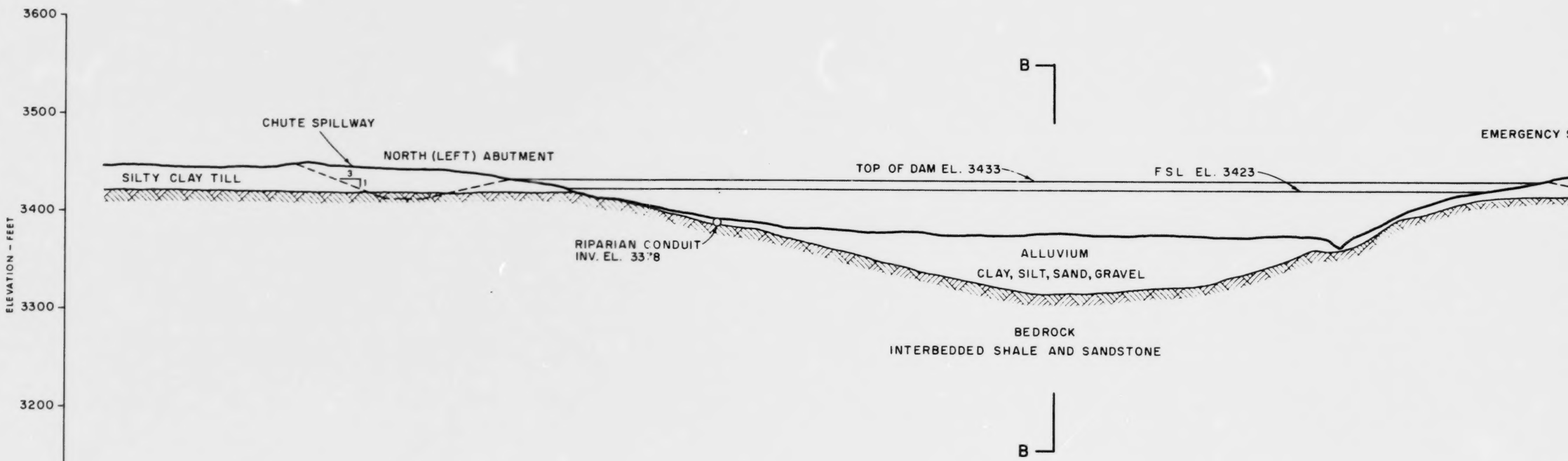
Drawing 5.1 Surficial Geology - Reservoir
Drawing 5.2 Surficial Geology - Damsite
Drawing 5.3 Geological Cross-Section
Drawing 5.4 Preliminary Embankment Design



////// INDICATES APPROXIMATE LIMITS OF RESERVOIR

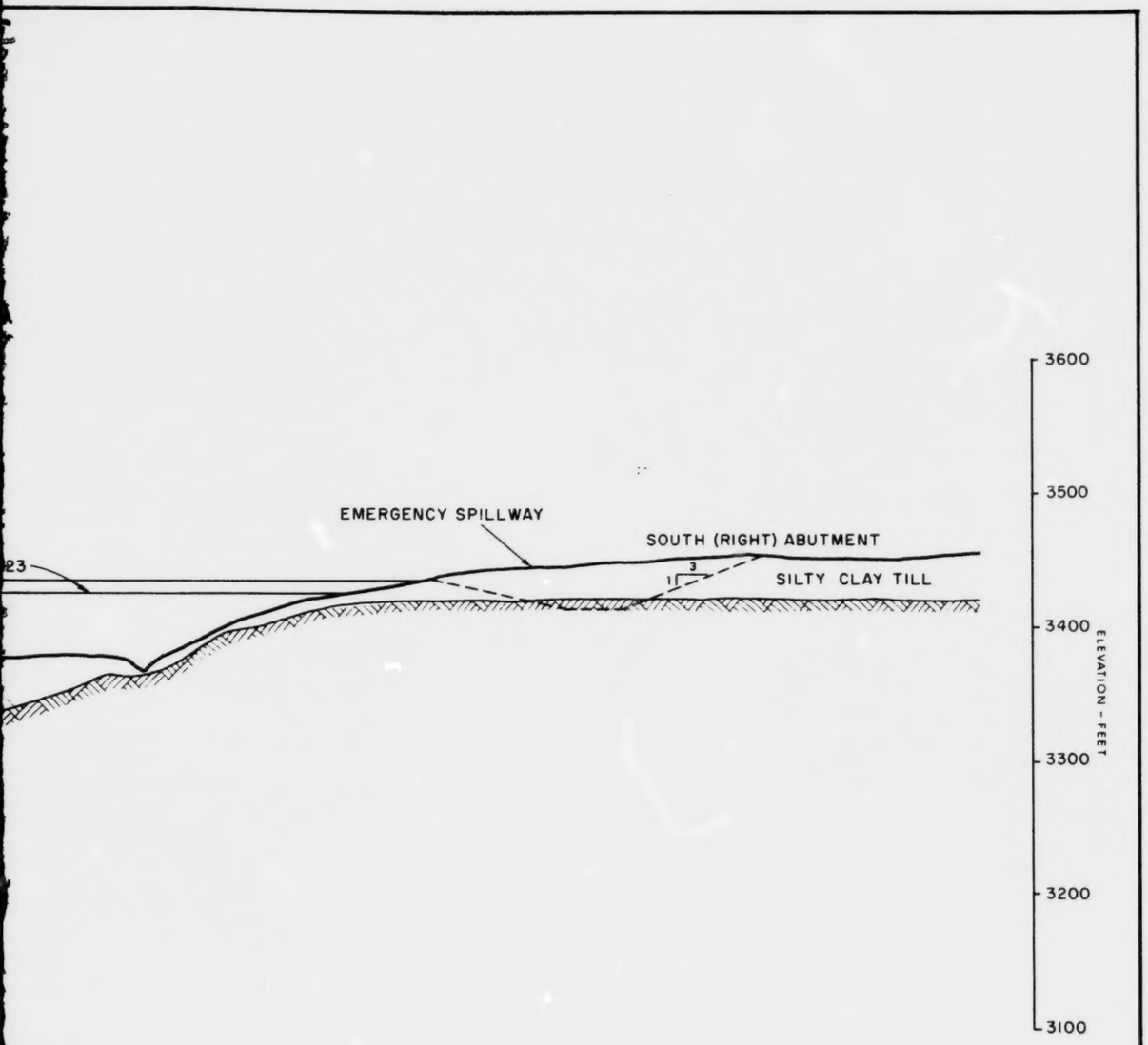
	W-E-R ENGINEERING LTD.	DRAWN IGJ/JAB
	SURFICIAL GEOLOGY MOSQUITO CREEK SITE 4 RESERVOIR	FILE NO. 19-293-0
		DATE MAY 1985
		APPROVED
		SCALE 1:60000
THURBER CONSULTANTS LTD., Geotechnical Engineers		DRAWING NO 5.1

	W-E-R ENGINEERING LTD.	DRAWN IGJ / JAB
	SURFICIAL GEOLOGY MOSQUITO CREEK SITE 4 DAMSITE	FILE NO. 19-293-0
		DATE MAY 1985
		APPROVED I Jones
		SCALE 1:15 000
THURBER CONSULTANTS LTD., Geotechnical Engineers		DRAWING NO 5.2

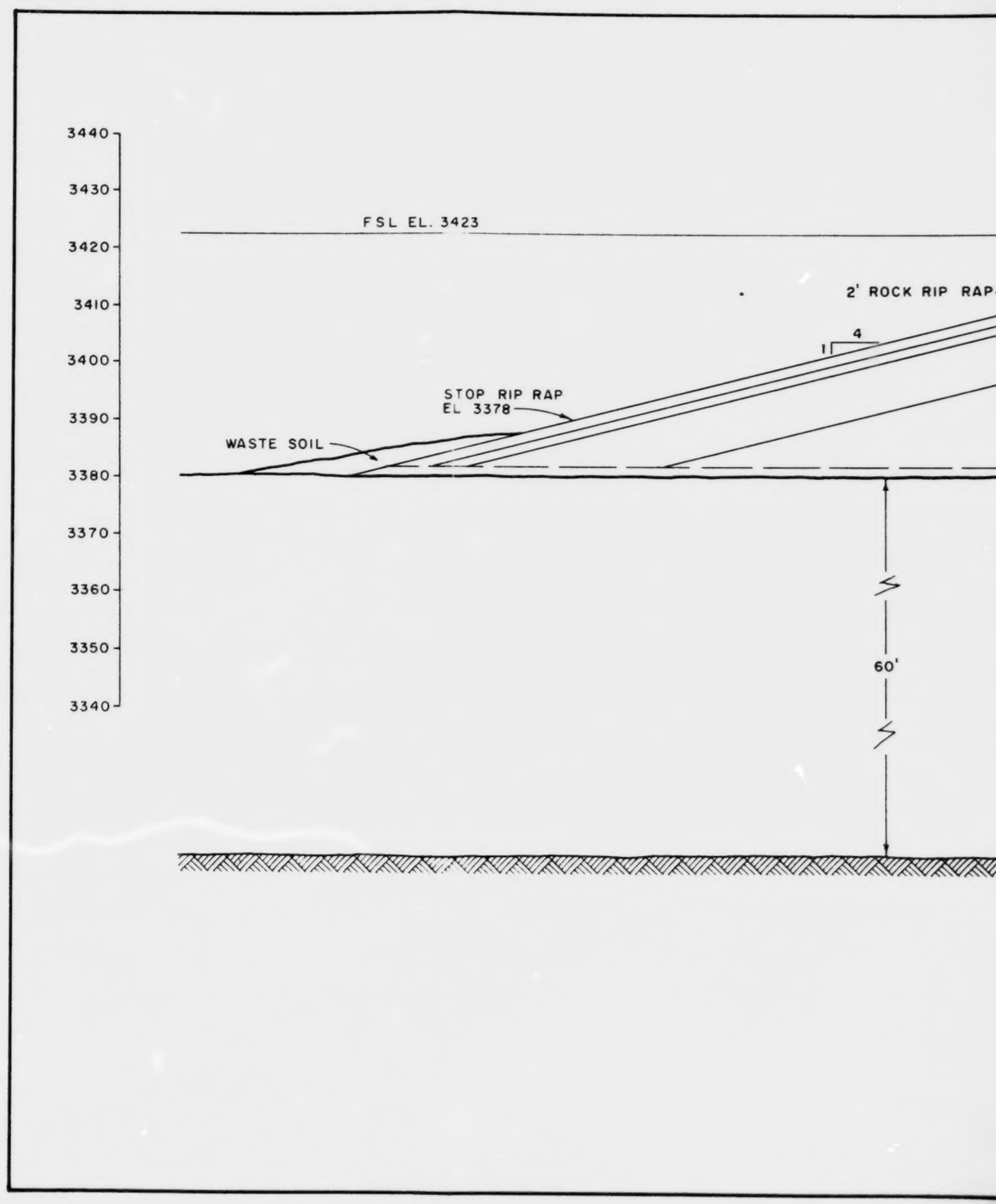


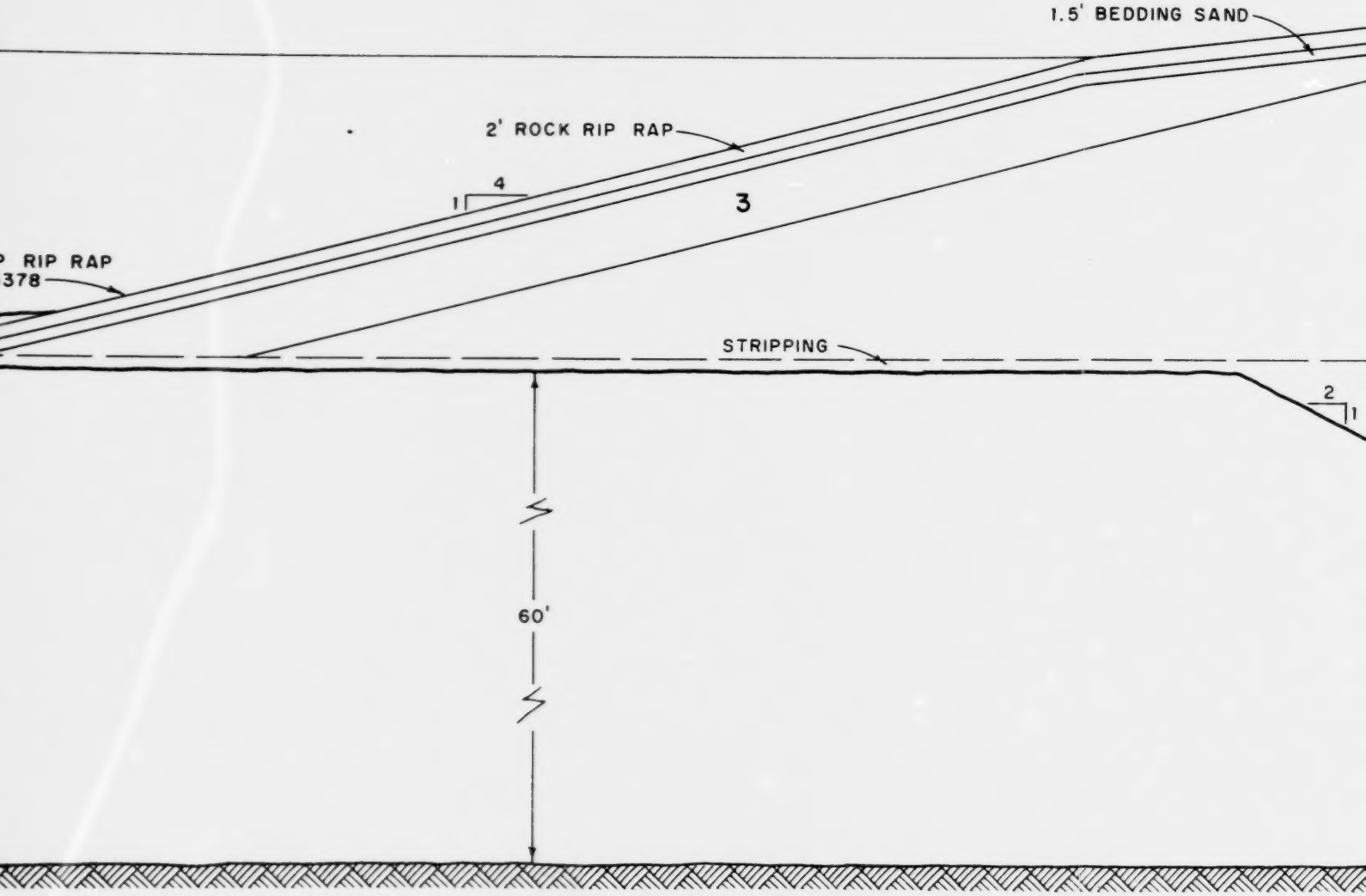
NOTES:
1. CROSS SECTION PREPARED FROM INFORMATION PRESENTED IN PFRA REPORT ENTITLED "PRELIMINARY SOIL MECHANICS REPORT, MOSQUITO CREEK PROJECT SITE 4", DATED SEPT. 12, 1967.
2. REFER TO DRAWING 5.4 FOR CROSS-SECTION B-B

THURBER CONSU



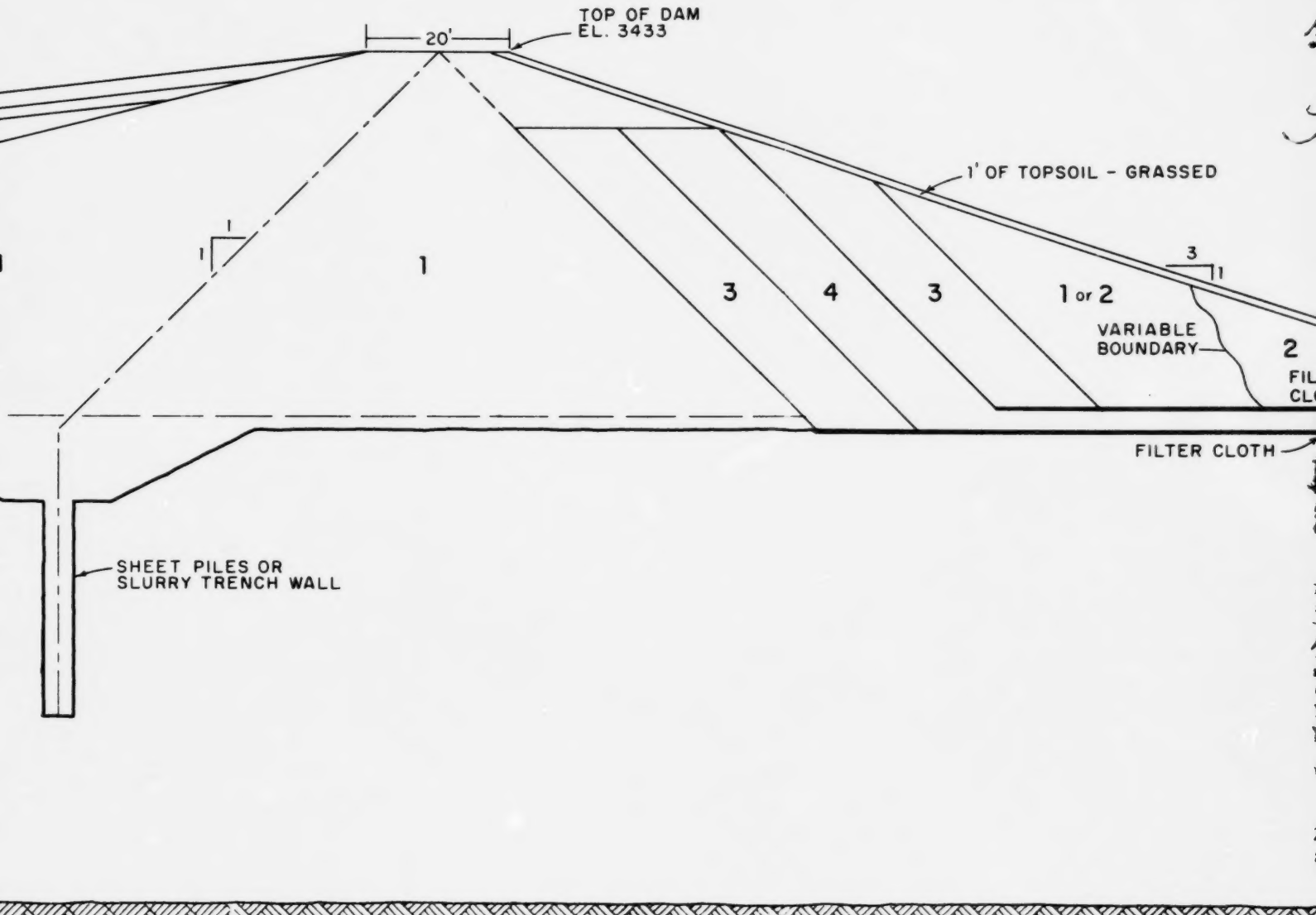
THUNDER CONSULTANTS LTD., Geotechnical Engineers	W-E-R ENGINEERING LTD.	DRAWN LBS / JAB
	GEOLOGICAL CROSS-SECTION MOSQUITO CREEK SITE 4	FILE NO. 19-293-0 DATE MAY 1985 APPROVED SCALE 1" = 100' DRAWING NO 5.3





1 IMPERVIOUS MATERIAL. WELL COMPACTED SILTY CLAY TILL FROM UPLANDS.

2 PERVIOUS TO SEMI-PERVIOUS MATERIAL. RANDOM SANDS AND GRAVELS FROM CUTOFF EXCAVATION OR FROM OTHER BORROW AREAS IN ALLUVIUM.



3 WELL GRADED SILT, SAND AND GRAVEL MIXTURE FROM BORROW AREAS IN ALLUVIUM.

4 PERVIOUS DRAIN. CLEAN SAND AND GRAVEL WITH LESS THAN 3 % PASSING THE NO. 8 SIEVE.

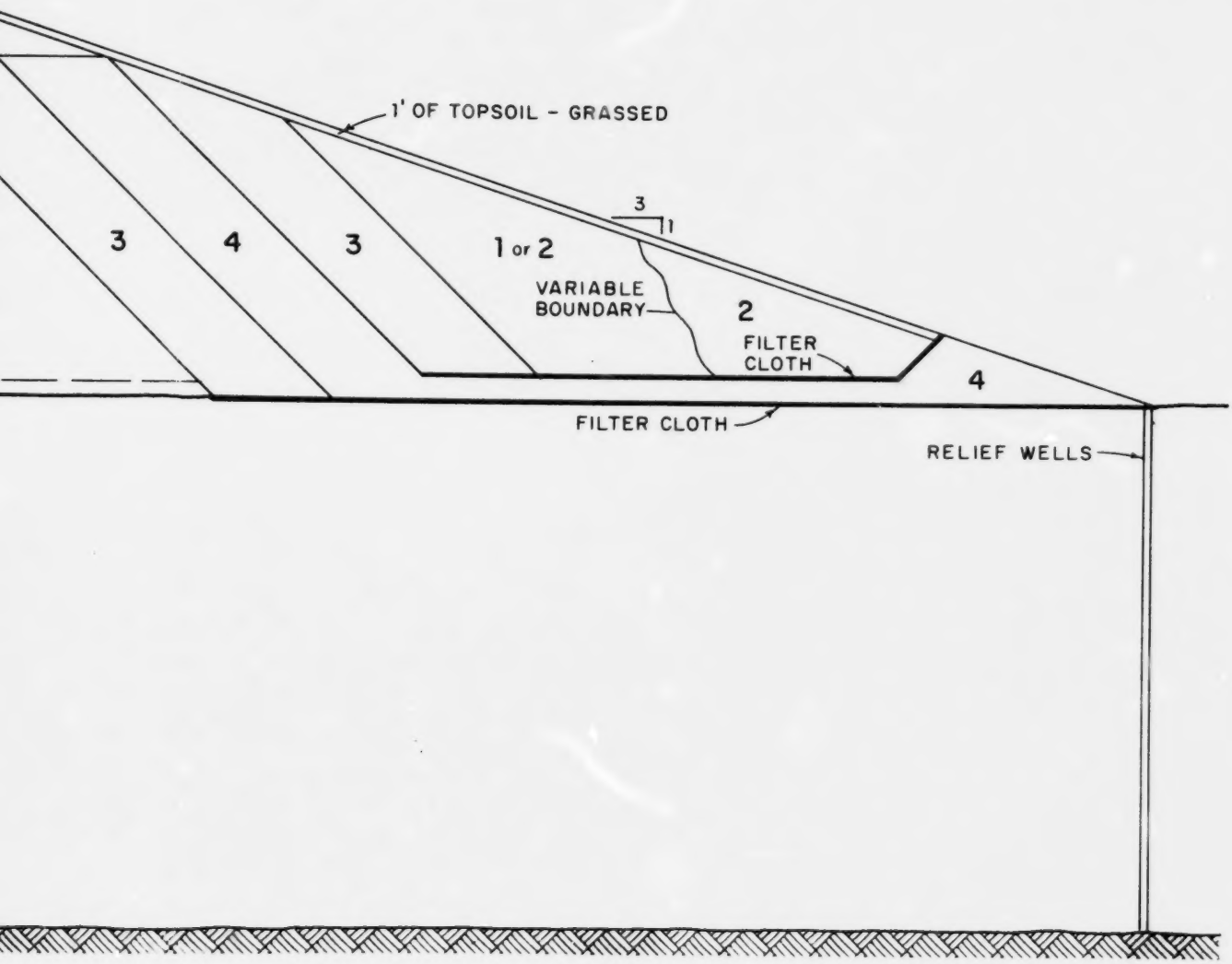
W-E-R ENGINEER

PRELIMINARY EMBA

MOSQUITO CRE

THURBER CONSULTANTS LTD.,

OF DAM
3433



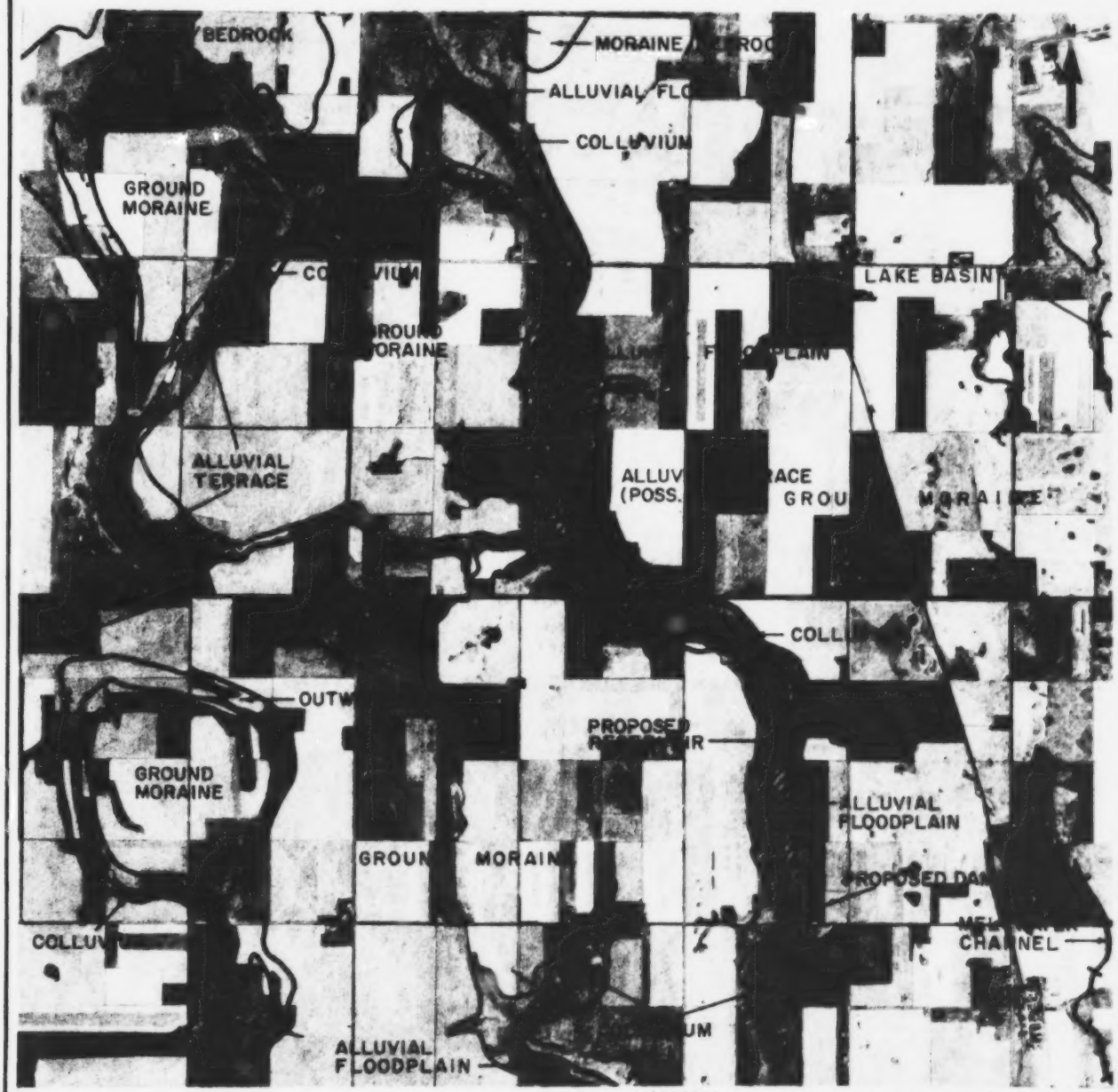
	W-E-R ENGINEERING LTD.	DRAWN LBS / JAB
	PRELIMINARY EMBANKMENT DESIGN MOSQUITO CREEK SITE 4	FILE NO 19-293-0 DATE MAY 1985 APPROVED SCALE 1" = 20'
	THURBER CONSULTANTS LTD., Geotechnical Engineers	DRAWING NO 5.4

APPENDIX C
TO GEOTECHNICAL REPORT

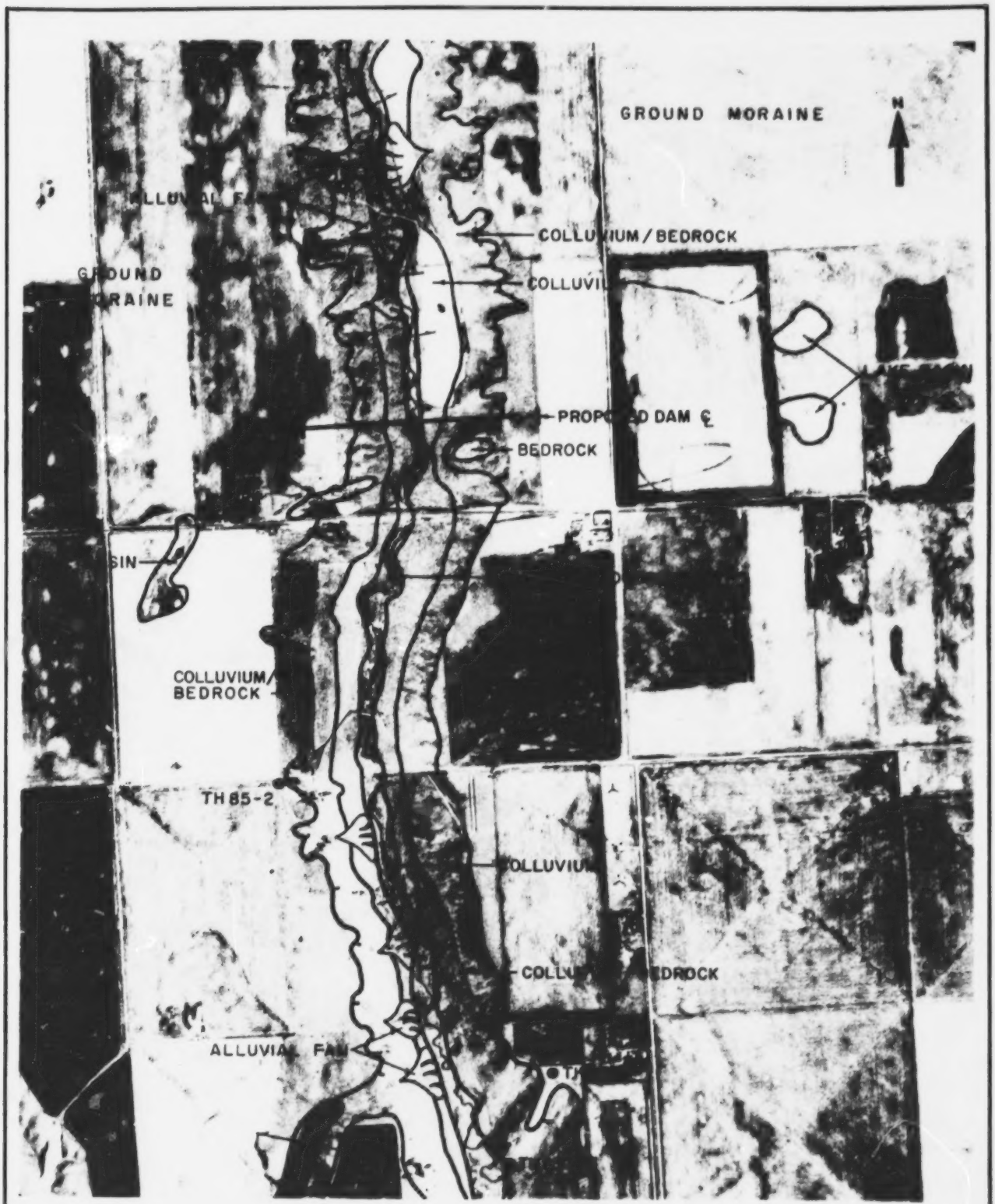
SQUAW COULEE SITE 2

Drawing 6.1 Surficial Geology - Reservoir
Drawing 6.2 Surficial Geology - Damsite
Drawing 6.3 Geological Cross-Section
Drawing 6.4 Preliminary Embankment Design

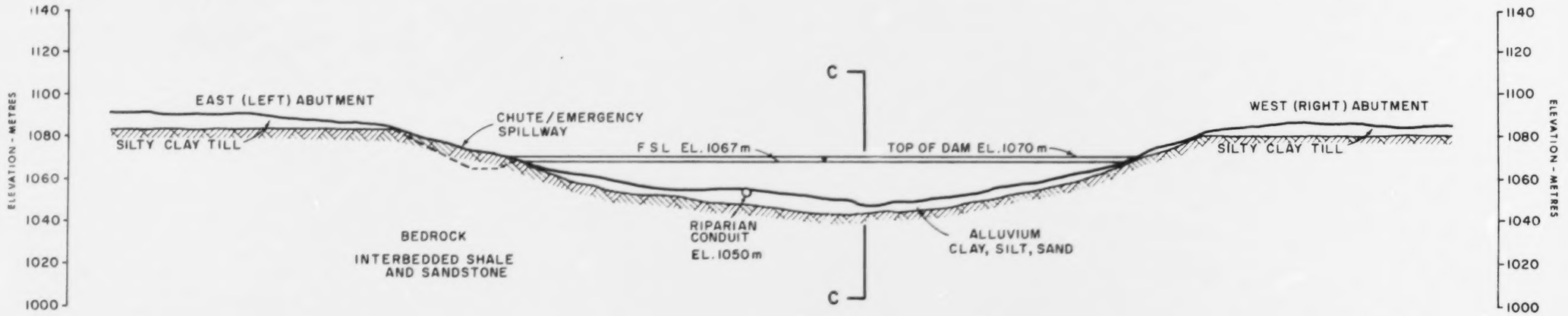
Test Hole Logs 85-2 to 85-4



	W-E-R ENGINEERING LTD.	DRAWN IGJ / JAB
	SURFICIAL GEOLOGY SQUAW COULEE SITE 2 RESERVOIR	FILE NO. 19-293-0
		DATE MAY 1985
		APPROVED Ifaues
		SCALE 1:60 000
THURBER CONSULTANTS LTD., Geotechnical Engineers		DRAWING NO 6.1



	W-E-R ENGINEERING LTD.	DRAWN IGJ/JAB
	SURFICIAL GEOLOGY SQUAW COULEE SITE 2 DAMSITE	FILE NO. 19-293-0
		DATE MAY 1985
		APPROVED
		SCALE 1:15 000
THURBER CONSULTANTS LTD., Geotechnical Engineers		DRAWING NO 6.2



NOTES:

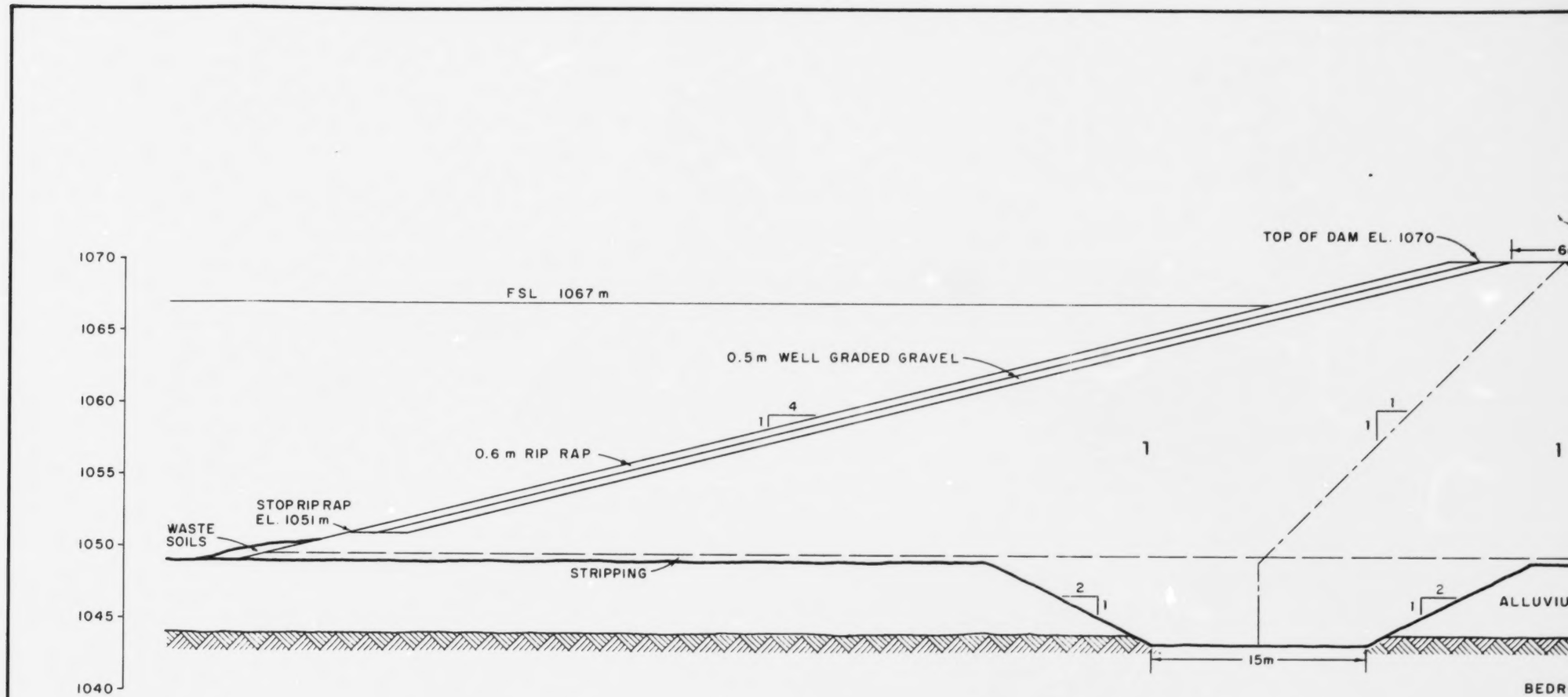
1. SUBSURFACE CONDITIONS AS SHOWN ARE BASED ON EXAMINATION OF AIRPHOTOS, FIELD INSPECTION AND SUBSURFACE CONDITIONS ENCOUNTERED IN THREE TEST HOLES DRILLED ABOUT 2 km DOWNSTREAM OF THIS SITE.
2. REFER TO DRAWING 6.4 FOR CROSS-SECTION C-C.

W-E-R ENGINEERING LTD.

GEOLOGICAL CROSS-SECTION
SQUAW COULEE SITE 2

THURBER CONSULTANTS LTD., Geotechnical Engineers

DRAWN	LBS / JAB
FILE NO.	19-293-0
DATE	MAY 1985
APPROVED	
SCALE	1 : 2000
DRAWING NO.	6.3

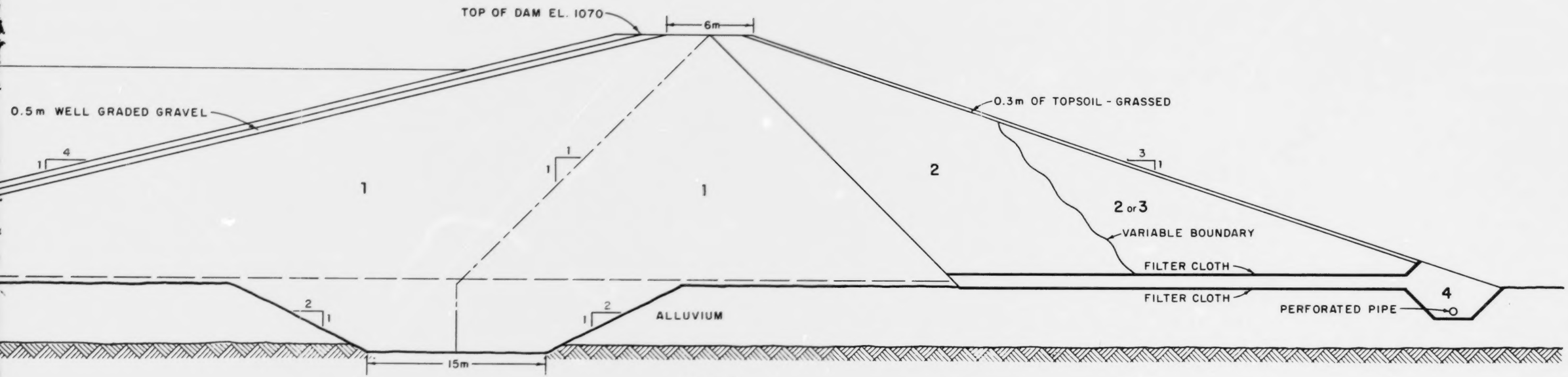


1 IMPERVIOUS MATERIAL. WELL COMPACTED SILTY CLAY TILL FROM UPLANDS.

2 PERVIOUS TO SEMI-PERVIOUS MATERIAL. RANDOM SANDS AND GRAVELS FROM CUTOFF EXCAVATION OR FROM BORROW AREAS IN ALLUVIUM.

3 PERVIOUS TO SEMI-PER
RECOMPACTED SHALE A
BEDROCK FROM SPILLW

4 PERVIOUS DRAIN. CLEA
WITH LESS THAN 3%
SIEVE.



1 IMPERVIOUS MATERIAL. WELL COMPACTED SILTY CLAY TILL FROM UPLANDS.

2 PERVIOUS TO SEMI-PERVIOUS MATERIAL. RANDOM SANDS AND GRAVELS FROM CUTOFF EXCAVATION OR FROM BORROW AREAS IN ALLUVIUM.

3 PERVIOUS TO SEMI-PERVIOUS MATERIAL. RECOMPACTED SHALE AND SANDSTONE BEDROCK FROM SPILLWAY EXCAVATION.

4 PERVIOUS DRAIN. CLEAN SAND AND GRAVEL WITH LESS THAN 3% PASSING THE NO. 8 SIEVE.

W-E-R ENGINEERING LTD.

PRELIMINARY EMBANKMENT DESIGN
SQUAW COULEE SITE 2

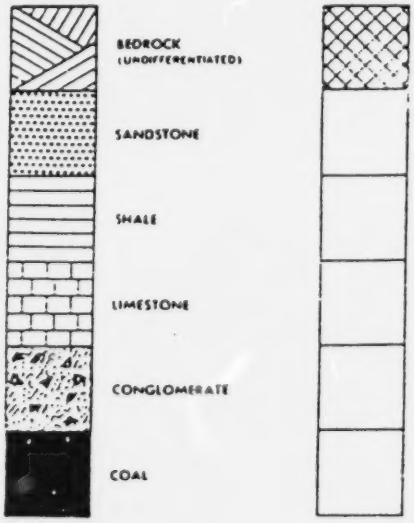
THURBER CONSULTANTS LTD., Geotechnical Engineers

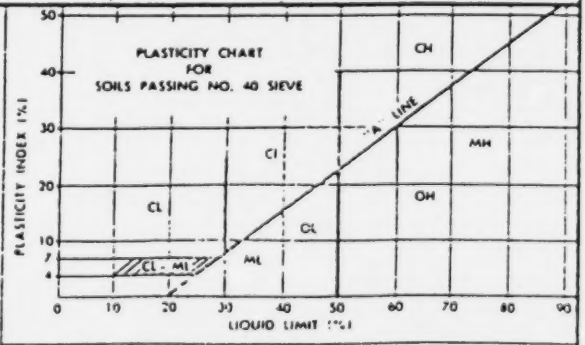
DRAWN	LBS/JAB
FILE NO.	19-293-0
DATE	MAY 1985
APPROVED	
SCALE	1:300
DRAWING NO.	6.4

MODIFIED UNIFIED CLASSIFICATION SYSTEM FOR SOILS

MAJOR DIVISION			GROUP SYMBOL	GRAPH SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA	
COARSE-GRAINED SOILS (MORE THAN HALF BY WEIGHT LARGER THAN 200 SIEVE)	GRAVELS (MORE THAN HALF COARSE FRACTION GRAINS LARGER THAN NO 4 SIEVE)	CLEAN GRAVELS (LITTLE OR NO FINES)	GW		WELL GRADED GRAVELS, LITTLE OR NO FINES	$C_U = \frac{D_{60}}{D_{10}} > 4 \quad C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
			GP		POORLY GRADED GRAVELS, AND GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		DIRTY GRAVELS (WITH SOME FINES)	GM		SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW "A" LINE OR P.I. LESS THAN 4
			GC		CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE "A" LINE P.I. MORE THAN 7
	SANDS (MORE THAN HALF COARSE FRACTION GRAINS SMALLER THAN NO 4 SIEVE)	CLEAN SANDS (LITTLE OR NO FINES)	SW		WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$C_U = \frac{D_{60}}{D_{10}} > 6 \quad C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}} = 1 \text{ to } 3$	
			SP		POORLY GRADED SANDS, LITTLE OR NO FINES	NOT MEETING ABOVE REQUIREMENTS	
		DIRTY SANDS (WITH SOME FINES)	SM		SILTY SANDS, SAND-SILT MIXTURES	CONTENT OF FINES EXCEEDS 12%	ATTERBERG LIMITS BELOW "A" LINE P.I. LESS THAN 4
			SC		CLAYEY SANDS, SAND-CLAY MIXTURES		ATTERBERG LIMITS ABOVE "A" LINE P.I. MORE THAN 7
FINE-GRAINED SOILS (MORE THAN HALF BY WEIGHT PASSES 200 SIEVE)	SILTS (BELOW "A" LINE NEGLIGIBLE ORGANIC CONTENT)	$W_L < 50\%$	ML		INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY SANDS OF SLIGHT PLASTICITY	CLASSIFICATION IS BASED UPON PLASTICITY CHART (see below)	
		$W_L > 50\%$	MH		INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS, FINE SANDY OR SILTY SOILS		
	CLAYS (ABOVE "A" LINE ON PLASTICITY CHART NEGLIGIBLE ORGANIC CONTENT)	$W_L < 30\%$	CL		INORGANIC CLAYS OF LOW PLASTICITY, GRAVELLY, SANDY, OR SILTY CLAYS, LEAN CLAYS		
		$30\% < W_L < 50\%$	CI		INORGANIC CLAYS OF MEDIUM PLASTICITY, SILTY CLAYS		
		$W_L > 50\%$	CH		INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS		
	ORGANIC SILTS & CLAYS (BELOW "A" LINE ON CHART)	$W_L < 50\%$	OL		ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	WHENEVER THE NATURE OF THE FINE CONTENT HAS NOT BEEN DETERMINED, IT IS DESIGNATED BY THE LETTER "F", E.G. SF IS A MIXTURE OF SAND WITH SILT OR CLAY	
		$W_L > 50\%$	OH		ORGANIC CLAYS OF HIGH PLASTICITY		
HIGHLY ORGANIC SOILS			Pt		PEAT AND OTHER HIGHLY ORGANIC SOILS	STRONG COLOR OR ODOR, AND OFTEN FIBROUS TEXTURE	

SPECIAL SYMBOLS





1. ALL SIEVE SIZES MENTIONED ON THIS CHART ARE U.S. STANDARD, A.S.T.M. E.11
2. BOUNDARY CLASSIFICATIONS POSSESSING CHARACTERISTICS OF TWO GROUPS ARE GIVEN COMBINED GROUP SYMBOLS, E.G. GW-GC IS A WELL GRADED GRAVEL SAND MIXTURE WITH CLAY BINDER BETWEEN 5% AND 12%.

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Geotechnical Engineers
EDMONTON VANCOUVER VICTORIA

SYMBOLS AND TERMS USED ON TEST HOLE LOGS

1. VISUAL TEXTURAL CLASSIFICATION OF MINERAL SOILS

CLASSIFICATION	APPARENT PARTICLE SIZE
Boulders	Greater than 200mm
Cobbles	75mm to 200mm
Gravel	5mm to 75mm
Sand	Not visible to 5mm
Silt	Non-Plastic particles, not visible to the naked eye
Clay	Plastic particles, not visible to the naked eye

2. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	APPROXIMATE UNDRAINED SHEAR STRENGTH	
Very Soft	Less than 10kPa	
Soft	10 - 25kPa	
Firm	25 - 50kPa	
Stiff	50 - 100kPa	
Very Stiff	100 - 200kPa	Modified from National Building Code
Hard	200 - 300kPa	
Very Hard	Greater than 300kPa	

3. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	STANDARD PENETRATION TEST (Number of Blows per 300mm)	
Very Loose	0 - 4	
Loose	4 - 10	
Compact	10 - 30	
Dense	30 - 50	Modified from National Building Code
Very Dense	Over 50	

4. LEGEND FOR TEST HOLE LOGS

- ○ Water content (% by weight) as determined on disturbed samples.
- ● Water content (% by weight) as determined on undisturbed samples.
- Disturbed bag or split spoon sample.
- Undisturbed Shelby Tube sample or core from VTM core barrel.
- No recovery
- Number of blows per 300mm for STANDARD PENETRATION TEST.
- C_u Undrained Shear Strength determined by unconfined compression test.
- C_{vane} Shear Strength determined by pocket vane.
- C_{pen} Shear Strength determined by pocket penetrometer.
- ▼ Water Level.

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Sheet 1 of 2

TEST HOLE № 85-2

LOG OF TEST HOLE

LOCATION West abutment, some 1000 m upstream from centre line

CLIENT W.E.R. Engineering Ltd.
PROJECT Little Bow River Basin Study
DATE March 14, 1985
METHOD Auger/Core
DRILLING CO Mobile Augers Ltd.
INSPECTOR C. Minogue

LEGEND:
SAMPLES: Disturbed, Undisturbed, No recovery
WATER CONTENT:
WATER LEVEL
w_L LIQUID LIMIT
w_P PLASTIC LIMIT

WATER CONTENT (%) 10 20 30 40 50

ELEVATION (METRES)

REMARKS

TOP OF HOLE ELEV.

UNIFIED SOIL CLASSIFICATION SYSTEM

LOG

DEPTH (METRES)

ML

TOPSOIL silt and sand, organic, brown

SILT sandy, light brown, dry to damp

CL-CI

N=10/22/26

CLAY (TILL) silty, some fine to medium sand, trace gravel, very stiff to hard, low to medium plastic, brown, damp, gypsum crystals

- - medium plastic, grey-brown

CI

- - sand pockets

N=12/31/34

SANDSTONE fine to medium, light brown, hard

Coring commenced at 8.8 m; no groundwater encountered prior to coring

Run 1: R=95%
R.Q.D.=46%

Run 2: R=95%
R.Q.D.=43%

SANDSTONE fine grained, poorly cemented, brown, weathered

- - cemented, "salt and pepper" texture

SCALE: 1:50

FILE № 19-233-0

THURBER CONSULTANTS LTD.
Geotechnical Engineers

Sheet 2 of 2

TEST HOLE Nº 85-2

LOG OF TEST HOLE

LOCATION West abutment, some 1000 m upstream from centre line

CLIENT W.E.R. Engineering Ltd.
PROJECT Little Bow River Basin Study
DATE March 14, 1985
METHOD Auger/Core
DRILLING Cº Mobile Augers Ltd.
INSPECTOR C. Minogue

LEGEND:
SAMPLES: Disturbed; Undisturbed; No recovery
WATER CONTENT: ▼ WATER LEVEL; W_L LIQUID LIMIT; W_P PLASTIC LIMIT

WATER CONTENT (%) 10 20 30 40 50

ELEVATION (METRES) — TOP OF HOLE ELEV.

REMARKS

UNIFIED SOIL CLASSIFICATION SYSTEM

LOG

DEPTH (METRES)

Run 2

Run 3: R=100%
R.Q.D.=52%

Run 4: R=100%
R.Q.D.=75%

@ 13.41-13.56 m core is poorly cemented and broken up

Run 5: R=105%
R.Q.D.=75%

Run 6: R=96%
R.Q.D.=41%

Run 7: R=91%
R.Q.D.=36%

10 SANDSTONE (continued)

11 MUDSTONE with SILTSTONE interbeds
- mudstone: silty, black to olive-green, weathered, locally nuggetty
- siltstone: muddy, green to grey

12 SILTSTONE
muddy, dark green-grey, occas. mud lens

13 more competent

14 MUDSTONE
silty, locally nuggetty, occas. siltstone stringer, to 25mm

15

16 SILTSTONE
muddy, dark green-grey, occas. mud lens to 75 mm

17 SANDSTONE
fine to very fine, organic laminations, some thin siltstone lenses, grey
silty, less well cemented, micaceous

18 End of hole at 18.0 m
Hole grouted in at completion

19

20

SCALE: 1:50 FILE Nº 19-293-0

THURBER CONSULTANTS LTD.
Geotechnical Engineers

Sheet 1 of 2

TEST HOLE Nº 85-3

LOG OF TEST HOLE

LOCATION East abutment, close to centre line

CLIENT W.E.R. Engineering Ltd.
PROJECT Little Bow River Basin Study
DATE March 15-16, 1985
METHOD Auger/Coring
DRILLING Cº Mobile Augers Ltd.
INSPECTOR C. Minogue

LEGEND:
SAMPLES: Disturbed, Undisturbed, No recovery
WATER CONTENT:
WATER LEVEL
W_L LIQUID LIMIT
W_P PLASTIC LIMIT

WATER CONTENT (%) 10 20 30 40 50

ELEVATION (METRES)

REMARKS

TOP OF HOLE ELEV.

UNIFIED SOIL CLASSIFICATION SYSTEM

LOG

DEPTH (METRES)

		TOPSOIL sand and silt, organic, brown	0
N=16/20/22	CL	SILT sandy, light brown, dry to damp	
		CLAY (TILL) silty, some fine sand, low plastic, brown, damp to dry	1
		damp to moist, oxide specks and salt lenses	2
N=12/22/- (hit rock)	CL		3

SCALE: 1:50 FILE Nº 19-293-0

THURBER CONSULTANTS LTD.
Geotechnical Engineers

Sheet 2 of 2

TEST HOLE NO 85-3

LOG OF TEST HOLE

LOCATION East abutment, close to centre line

CLIENT W.E.R. Engineering Ltd.

PROJECT Little Bow River Basin Study

DATE March 15-16, 1985

METHOD Auger/Coring

DRILLING Co Mobile Augers Ltd.

INSPECTOR C. Minogue

LEGEND:

SAMPLES: Disturbed; Undisturbed; No recovery

WATER CONTENT:

WL LIQUID LIMIT

WP PLASTIC LIMIT

WATER LEVEL

WATER CONTENT (%) 10 20 30 40 50	ELEVATION (METRES)	REMARKS	UNIFIED SOIL CLASSIFICATION SYSTEM	LOG	DEPTH (METRES)
	TOP OF HOLE ELEV.	Coring commenced at 5.2 m; no groundwater encountered prior to coring; Run 1: R=95% R.Q.D.=100%	SM	SANDSTONE fine grained, yellow-brown, cemented	
		@ 6.12 m: bedding plane fracture, planar, smooth, weathered Run 2: R=94% R.Q.D.=46%			
		@ 6.40-6.55 m: sub-vertical fracture, planar, smooth, weathered		-- less well cemented, damp	7
		Run 3: R=76% R.Q.D.=19%		-- pebble layer	8
				--cross-bedded, cemented	
		Run 4: R=71% R.Q.D.=24%		-- mudstone inclusions, less well cemented	9
				End of hole at 9.3 m Refusal in sandstone Hole grouted at completion	

SCALE: 1:50

FILE NO 19-293-0

THURBER CONSULTANTS LTD.
Geotechnical Engineers

TEST HOLE № 85-4

LOG OF TEST HOLE

LOCATION In bottom of coulee, close to centre line and downslope from TH SC2A-2

CLIENT W.E.R. Engineering Ltd.
PROJECT Little Bow River Basin Study
DATE March 16, 1985
METHOD Auger
DRILLING Co Mobile Augers Ltd.
INSPECTOR C. Minogue

LEGEND:
SAMPLES: □ Disturbed ■ Undisturbed ⊠ No recovery
WATER CONTENT: ○
▼ WATER LEVEL
W_L LIQUID LIMIT
W_P PLASTIC LIMIT

WATER CONTENT (%) 10 20 30 40 50	ELEVATION (METRES)	REMARKS	UNIFIED SOIL CLASSIFICATION SYSTEM	LOG	DEPTH (METRES)
	TOP OF HOLE ELEV.			TOPSOIL silt and sand, organic, dark brown	0
○			ML-CL	SILT sandy (fine), light brown, damp, salt lenses	1
		N=11/12/12			2
○				very sandy	3
○		N=8/8/9	CL	CLAY very silty, very sandy, trace gravel, low to non-plastic, firm to stiff, oxide lenses, brown, damp	4 5
○		N=48 for 10 cm	SM	SANDSTONE fine grained, hard, light brown	6
○			SM		
				End of hole at 6.4 m Auger refusal Dry at completion Hole grouted in at completion	7 8 9 10

SCALE: 1:50 FILE № 19-293-0

LITTLE BOW RIVER BASIN
STORAGE SITE INVESTIGATION
FEASIBILITY DESIGN REPORT
June, 1985

APPENDIX C

DETAILED SUMMARY OF DESIGN STUDIES

AND COST ESTIMATES

LIST OF TABLES

Title	Table No.
Detailed Cost Estimate - Little Bow 1	C1
Pertinent Data - Little Bow 1	C2
Detailed Cost Estimate - Mosquito Creek 4	C3
Pertinent Data - Mosquito Creek 4	C4
Detailed Cost Estimate - Squaw Coulee 2	C5
Pertinent Data - Squaw Coulee 2	C6

LIST OF DRAWINGS

Title	Drawing No.
Little Bow Site 1 - General Layout	C1
Mosquito Creek Site 4 - General Layout	C2
Squaw Coulee Site 2 - General Layout	C3

DESIGN OF MAIN WORKS

1.0 General

This section covers the structure layout, hydraulic and structural design, and costing aspects of the main dam and appurtenant works for each of the three damsites under consideration.

2.0 Available Data

These investigations utilized the hydrologic and reservoir sizing data as provided in Appendix A and the geotechnical assessment as presented in Appendix B, in support of designs for the main works.

Photogrammetric mapping from the original PFRA reports for the proposed Little Bow and Mosquito Creek Reservoirs, at scales of one inch to 1,000 feet (20 ft. contours) and one inch to 800 feet (10 ft. contours), respectively, were used as reservoir mapping for those alternatives. PFRA site plans for Little Bow Site 1 and Mosquito Creek Site 4, which had been prepared from field surveys at a scale of one inch to 200 feet (5 ft. contours) became the base mapping for site layout purposes. New photogrammetric mapping for Squaw Coulee Site 2 was produced at a scale of 1:10,000 (3 m contours) for the reservoir and 1:2,000 (2 m contour) for the damsite, tied to geodetic datum and utilizing ground control as required.

3.0 Basic Project Requirements

The following subsections outline the project requirements established as a basis for preparing the feasibility design and cost estimates of each of the three damsites.

3.1 Reservoir Sizing

Storage curves produced in the conceptual study for the Little Bow and Mosquito Creek sites were again utilized in this investigation. A new area-capacity curve was, however, generated for Squaw Coulee Site 2 using the newly produced topographic mapping.

Storage curves produced in the conceptual study for the Little Bow and Mosquito Creek sites were again utilized in this investigation. A new area-capacity curve was, however, generated for Squaw Coulee Site 2 using the newly produced topographic mapping. The previous conceptual design studies established that it would not be feasible to construct reservoirs at any of the three sites beyond the topographic or physical maximum. This was a limiting factor in the design of these storages, notwithstanding the fact that water supplies are available to support larger reservoirs. The topographic maximum, which roughly corresponds to the hydrologic optimum reservoir size as established in the water supply studies (Appendix A) was therefore used to establish FSL's for all three reservoir alternatives in the current study.

3.2 Freeboard

Freeboard above the full supply level (FSL) for each dam was calculated to include an allowance for wind setup, wave runup as well as appropriate allowances for gates and frost action.

3.3 Controlling Floods

Design flood hydrographs used in spillway sizing are as presented in Appendix A.

The Dam and Canal Safety Guidelines as stipulated by the Alberta Environment, Dam Safety Branch were strictly adhered to for this investigation. Consequently, the following design flood criteria were established for spillway sizing purposes:

- spillways sized to adequately handle the 1:100 (project design) flood routed outflow with approximately wave height freeboard on the dam
- in all cases, the spillways must capably pass the safety-of-dam flood (equivalent to 0.5 x PMF) with a minimum of one foot of freeboard remaining.

Construction diversion floods were set at the 1 in 10 summer storms.

3.4 Downstream Irrigation Requirement/Storage Utilization

Project requirements dictate that the outlet works be capable of releasing 300 cfs at minimum reservoir levels, for downstream irrigation purposes. The invert of the intake was set slightly below the floodplain level, at each site, to fully utilize the reservoir storage capability without encroaching on the live storage.

4.0 Site Layouts and Selection of Elements

4.1 General

The embankment centrelines at Little Bow Site 1 and Mosquito Creek Site 4 are identical to those proposed by PFRA. However, the dams have been raised to accommodate the larger storage called for in the current study. Also, the spillways have been relocated to better suit the higher reservoir

levels. Conduit locations selected by PFRA were still judged as most suitable in terms of foundation and topography.

At Squaw Coulee Site 2, the proposed embankment, spillway and conduit centrelines have been located primarily to accommodate the topography, geology and foundation conditions.

General layouts as shown in Figures C1, C2 and C3 represent the feasibility level structural studies as carried out in this investigation.

4.2 Embankments

Embankment designs used in this feasibility study are outlined in Appendix B. The embankments are intended to utilize materials readily available at each of the sites. The cost estimates presented herein, reflect the nature of the material excavated and the quantities of borrow required.

Foundation conditions at Mosquito Creek Site 4 require special attention as indicated in the geotechnical study and the measures proposed have been allowed for in cost estimating for that site.

The predicted embankment settlement at each damsite requires that crests be cambered to ensure maintenance of the required freeboard should such movements occur. Dam crest elevations shown in the structure layouts reflect the levels required to accommodate the selected reservoir sizes. Precise elevations along the crest (including camber) would be determined during final design. However, embankment quantities calculated for costing purposes include an allowance for this camber.

A road relocation across the dam at Mosquito Creek Site 4 (see Appendix D) which requires a driving surface on the dam have been incorporated into the design and cost of these works.

4.3 Spillways

a) Hydraulic Design

The spillways proposed at each site are gated chute type structures with hydraulic jump stilling basins. Spillways equipped with radial gates were selected as being the most cost effective. It has been assumed that gate operators will be available for the project, however, normal gate operation will be automated.

Spillway dimensions are based on optimization studies to determine the most economical combination of crest length and gate height to pass the required flood flows. Chutes are flared into the stilling basin to ensure that flow energy is adequately dissipated.

The spillways are designed to efficiently pass the routed 1:100 year flood (project design flood). The structures are also sized to handle floods up to one half the probable maximum flood (0.5 PMF) with the possibility of limited damage being sustained in the lower structural components of the spillway.

Basic spillway dimensions for each site may be found in Figures C1, C2 and C3, and Tables C3, C6 and C9.

The rating curves provided are based on an average weir coefficient of 2.10 (metric) with the gates fully opened at passage of flood.

b) Location

All structures have been generally located to take advantage of topographic and foundation conditions. Spillway profiles are configured to minimize excavation depths, although all three spillways would intercept bedrock layers and some degree of rebound is anticicpated. Approach channels have been aligned to ensure that high velocities do not endanger the upstream abutment of the dam. Avoidance of downstream erosion near the toe of the dam was the main objective in the exit channel alignment. Materials excavated from the spillways can be utilized in the main dam embankments at each damsite.

c) Structural Design

The spillway structures are of conventional reinforced concrete design, with structural components sized approximately by consideration of similar existing structures. The required space and supports for the radial gates, hoisting systems and upstream stoplogs are provided by adequately sized piers and abutments. Gate hoists, control and additional equipment would be located on hoisting decks spanning between the piers. In the case of Mosquito Creek, the deck would also act as a traffic bridge across the spillway to accommodate the required road relocation.

Conventional jointing, cutoffs and key walls have been incorporated into the design to allow for articulation should differential foundation movement occur. The proposed chute and stilling basin sections consist of cantilever side walls and interior floor slabs articulated from the walls. In the lower chute and basin areas, concrete thicknesses have been increased to withstand the large dynamic forces and ensure stability.

d) Foundation and Drainage

Foundation seepage will be controlled by means of cutoffs and cross-walls.

Where foundations in the region of the crest section are fissured or pervious, some seepage control will be necessary through drainage conduits and relief wells. For the chute and stilling basin sections a granular bedding layer is proposed for the purposes of seepage and uplift pressure control. In the chute area, seepage through the foundation will be relieved by pipe drains located directly under the slab with gravity outlets to drainage conduits located parallel to and outside the retaining walls. In the basin area where substantially greater hydrostatic uplift forces are possible a curtain wall and pressure relief pipes connected to the subfloor drainage system are proposed to control uplfit pressures.

4.4 Irrigation Conduit

Irrigation conduits will consist of precast concrete pipelines with a reinforced concrete inlet structure, gatewell and outlet structures. Control of flow will be achieved by means of the gatewell which will be a two-celled structure located just upstream of the dam centreline, equipped with service and auxiliary gates, complete with hoisting equipment.

Details of these conduits, including basic dimensions are shown in Figures C1, C2 and C3 and Tables C3, C6 and C9.

The conduits would be located where bedrock is relatively shallow in order to take advantage of sound foundations and minimized excavation. Structure locations for Little Bow Site 1 and Mosquito Creek 4 correspond to those used in the PFRA studies.

To ensure maximized live storage, irrigation conduits have been designed for a release of 300 cfs at minimum reservoir levels. The outlet structure will be a hydraulic jump basin designed to handle the full gate, maximum head condition. Conduits will be equipped with air supply piping to prevent gate cavitation. Gradients have been set to minimize slug flow or other adverse hydraulic action.

Conduits will also be utilized for construction diversion purposes and converted to permanent irrigation works at completion of the project.

5.0 Basis of Costing

Detailed cost estimates for the proposed works can be found in Tables C2, C5 and C8. These estimates are intended to include all costs associated with construction of the dam embankments and ancilliary works, including contingencies. Allowances for engineering, design and surveys have also been included. However, interest during construction and annual charges have been omitted.

Unit prices utilized in the preparation of cost estimates for this study are based on bid prices from recent tenders on similar storage projects, as well as current tender prices on major irrigation works, in Southern Alberta.

The intended scope and/or coverage of specific items which appear in the detailed cost estimate tables are as follows:

1. Care of Water - is assumed to cover the cost of miscellaneous works for dewatering the site area during construction, but does not include the cost of major items for river or creeks diversion such as conduits and cofferdams.

2. Excavations - are assumed to cover all costs necessary to excavate and haul material from its source to place of deposition. Costs include such work as stripping, trimming, ripping, etc.

3. Embankments - are assumed to cover all costs necessary to place and compact material in the embankment or backfill areas. Costs include allowances for mixing material, adding, moisture, etc.

4. Embankment Filter and Drainage System - is assumed to cover all costs necessary to supply and place downstream filter and drainage systems including perforated pipes and relief wells.

5. Rock Riprap - is assumed to cover all costs necessary to harvest, process, haul and place riprap.

6. Granular Blanket Below Riprap - is assumed to cover all costs necessary to excavate, process, haul and place materials on the embankment.

7. Shotcrete Abutment Treatment - is assumed to cover all costs necessary to prepare the abutment adjacent to the impervious clay core, including shotcreting.

8. Concrete Items - are assumed to cover all costs necessary to produce and place concrete as shown on the drawings. Costs include the supply and placement of reinforcement steel and formwork, processing of aggregates and finishing.

9. Spillway Drainage System - is assumed to include all costs associated with the subfloor drainage system under the spillway chute and stilling basin and the conduit outlet basin.

10. Gate Items - are assumed to cover all costs necessary to manufacture and install complete working gates including hoisting systems.

11. Miscellaneous Items - are assumed to cover items which are anticipated at this time as being required, but which are too small or too detailed to include in the cost table.

12. Contingencies - are assumed to cover items of unknown or extra work which cannot be anticipted at this time because of the preliminary nature of the study and the lack of detailed background or field information.

LITTLE BOW SITE 1

TABLE C1
DETAILED COST ESTIMATE
LITTLE BOW SITE 1

	Quantity	Unit Price	Extension
Care of Water	lump sum		219,000
SUBTOTAL			
Embankment			
Foundation preparation			
- Stripping	21,900 yds^3	1.25	27,375
- Key Trench Excavation	55,500 yds^3	2.00	111,000
- Abutment - shotcreting	175 yds^3	475.00	83,125
Compacted Embankment			
- Zone #1	423,400 yds^3	0.70	296,380
- Zone #2	164,000 yds^3	0.70	114,800
Borrow Excavation	703,600 yds^3	1.50	1,055,400
Embankment Filter and Drainage System			
- Zone #4	25,800 yds^3	16.00	412,800
- Filter Cloth	37,100 yds^2	1.25	46,375
Rock Riprap	22,400 yds^3	27.00	604,800
Riprap Bedding	17,600 yds^3	12.00	211,200
Topsoil D/S Face of Dam	9,300 yds^3	1.25	11,625
SUBTOTAL EMBANKMENT			2,975,000
Spillway			
Excavation and Haul			
- Rock	22,850 yds^3	5.00	114,250
- Other	89,250 yds^3	2.00	178,500
Zone #1 Backfill	20,000 yds^3	3.00	60,000
Reinforced Concrete	1,019 yds^3	400.00	407,600
Drainage System	301 yds^3	19.00	5,700
Radial Gates & Hoists (2 sets)	lump sum		68,300
Miscellaneous Items			41,730
SUBTOTAL SPILLWAY			876,000
Irrigation Supply Conduit and Outlet Works			
Excavation - Structural	2,100 yds^3	2.00	4,200
Inlet and Outlet Channel Excavation	5,700 yds^3	2.00	11,400
Zone #1 Backfill	1,500 yds^3	3.00	4,500
Reinforced Concrete Pipe	370 lin. ft.	380.00	140,600
Structural Concrete	490 yds^3	600.00	294,000
Control Gates & Hoists	lump sum		51,700
Structure Drainage Systems	38 yds^3	19.00	700
Miscellaneous Items			25,000
SUBTOTAL IRRIGATION SUPPLY CONDUIT OUTLET WORKS			532,000
SUBTOTAL CONSTRUCTION			4,602,000
Contingencies (20%)			920,000
TOTAL CONSTRUCTION COST			5,522,000

TABLE C2
Pertinent Data
Little Bow Site 1

Reservoir

Full Supply Level (FSL)	3,290 ft.
Capacity at FSL	17,250 ac.ft.
Flooded Area at FSL	810 ac.
Maximum Depth at FSL	5[illegible] ft.
Probable Maximum Flood (inflow)	7,200 cfs
Project Design Flood - 0.5 PMF (inflow)	3,600 cfs
Maximum Water Level	3,295.6 ft.

Embankment

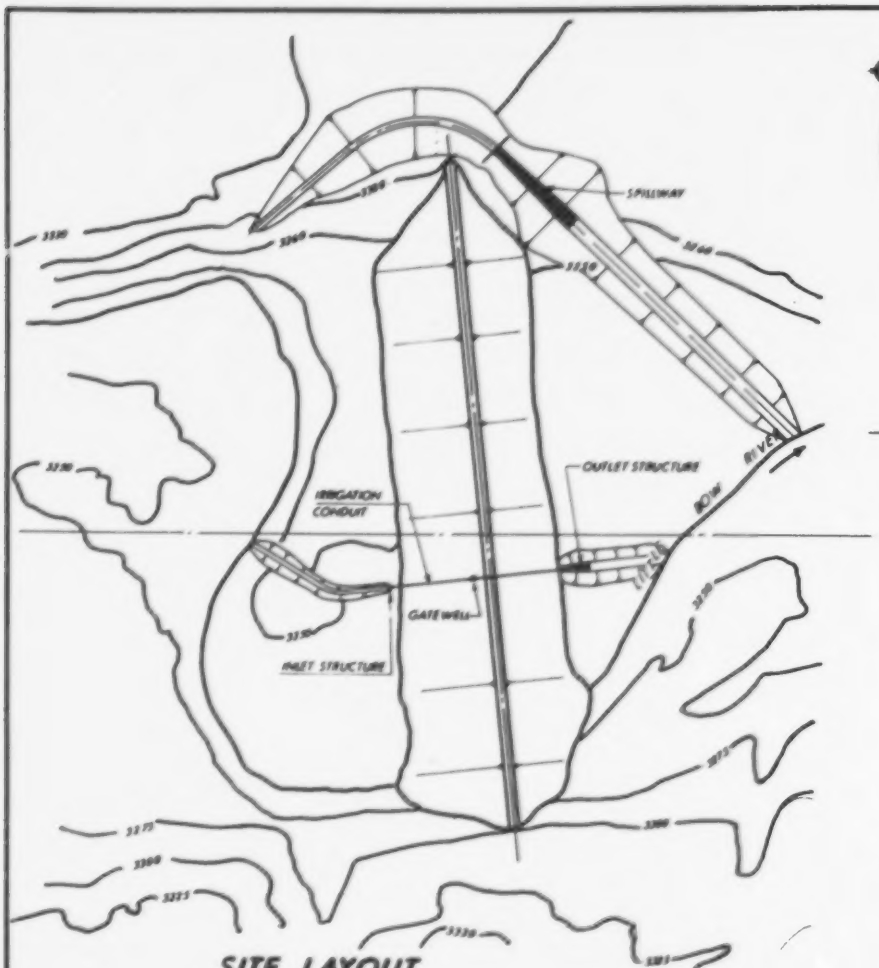
Top of Dam Elevation	3,299 ft.
Crest Length	820 ft.
Maximum Height	65 ft.
Volume of Fill	662,000 $yds.^3$
Freeboard at Design Discharge	7.6 ft.
Freeboard at Probable Maximum Discharge	3.4 ft.

Spillway

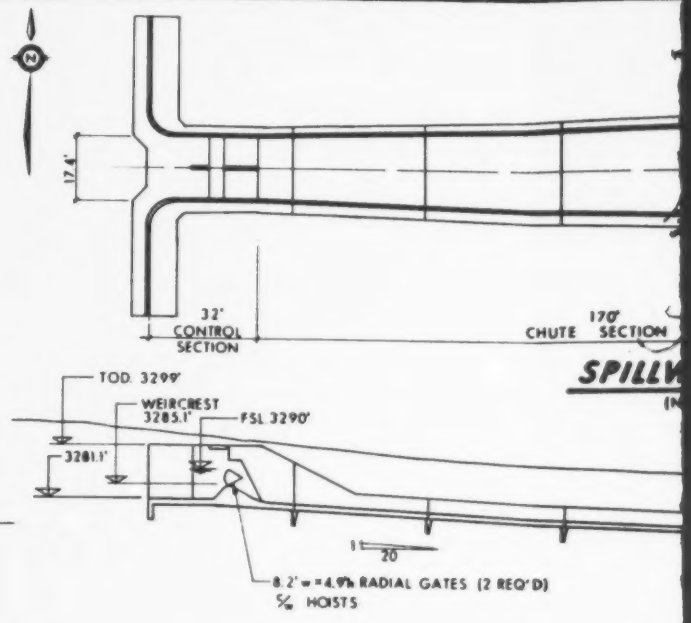
Crest Elevation	3,285.1 ft.
Radial Gates	8.2' W x 4.9' H
Net Weir Length	16.4 ft.
Gross Crest Width	17.4 ft.
Basin Width	32 ft.
Structure Length	246 ft.
Basin Elevation	3,234 ft.
Design Discharge	1,000 cfs
Probable Maximum Discharge	2,160 cfs

Irrigation Supply Conduit

Size of Conduit	5 ft. dia.
Length	380 ft.
Irrigation Supply Capacity Requirement	300 cfs
Gate Size	6' x 6'
Intake Elevation	3,243 ft.
Basin Elevation	3,234 ft.
Minimum Reservoir Elevation to Pass Irrigation Requirement	3,257 ft.

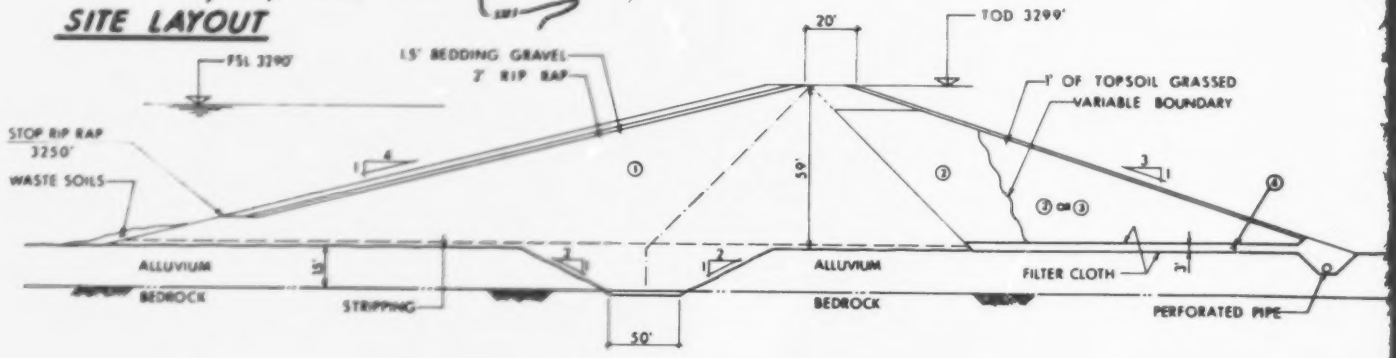


SITE LAYOUT



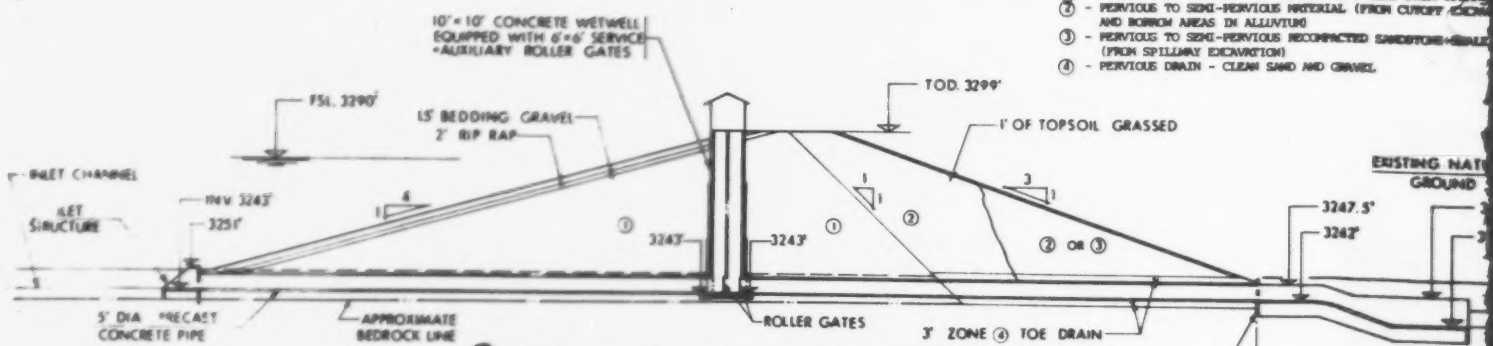
SPILLW

℄ SECTIO



MAXIMUM EMBANKMENT SECTION

EMBANKMENT ZONES
① - IMPERVIOUS MATERIAL (COMPACTED SILTY CLAY FROM UPL
② - PERVIOUS TO SEMI-PERVIOUS MATERIAL (FROM CUTOFF
AND BORROW AREAS IN ALLUVIUM)
③ - PERVIOUS TO SEMI-PERVIOUS RECOMPACTED SANDSTONE
(FROM SPILLWAY EXCAVATION)
④ - PERVIOUS DRAIN - CLEAN SAND AND GRAVEL



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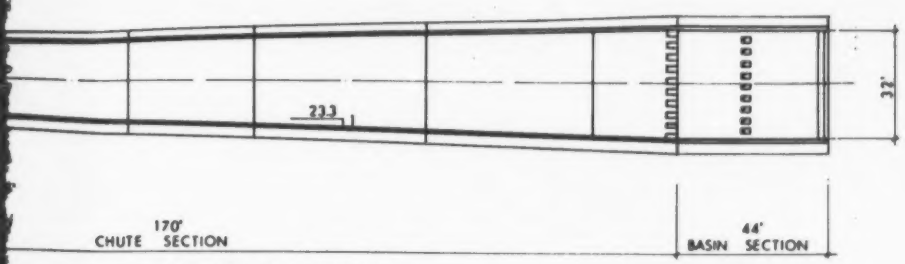
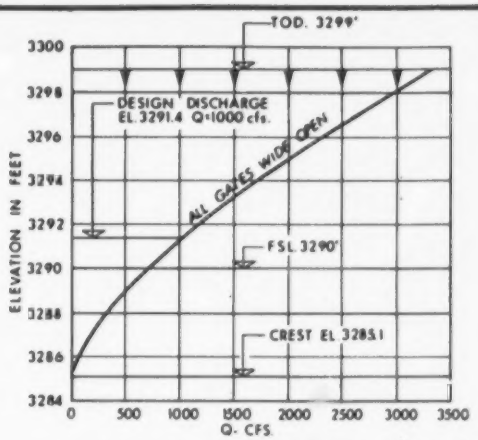


SPILLWAY PLAN
(NTS)



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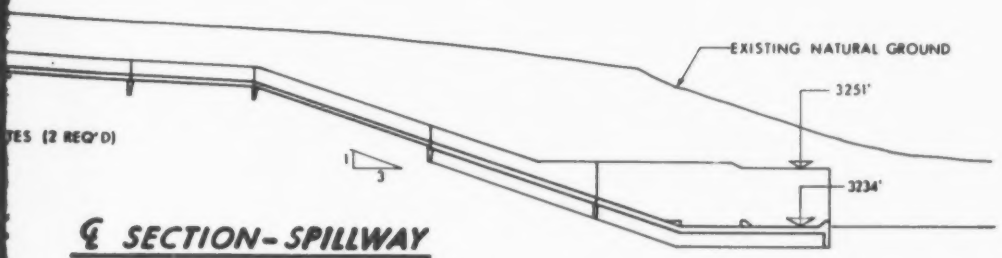

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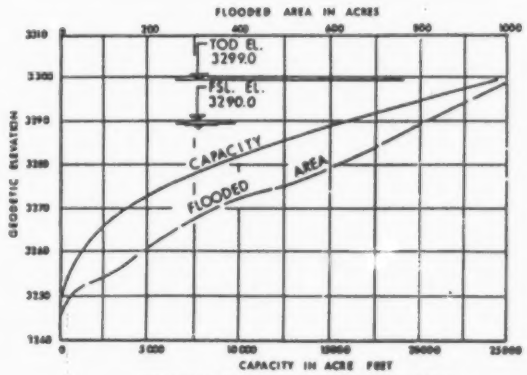
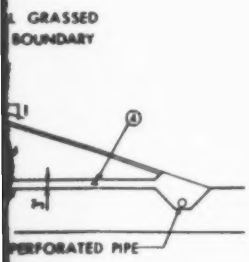
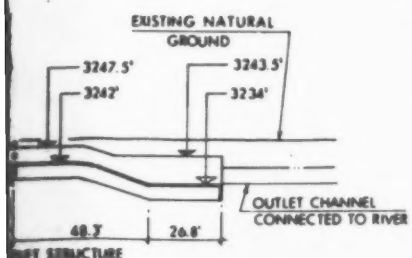
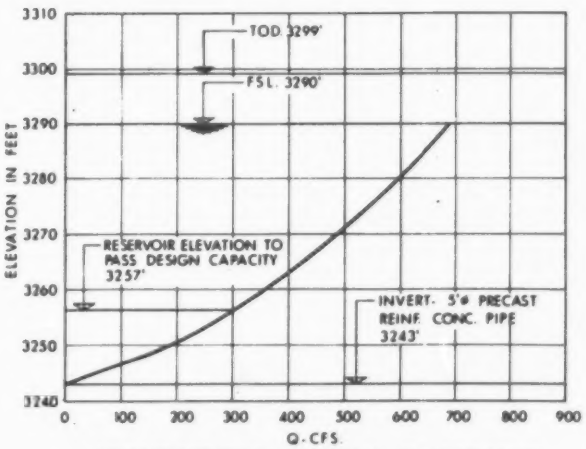


FLOODED AREA AND CAPACITY CURVES







IRRIGATION SUPPLY CONDUIT RATING CURVE

NOTE: ALL ELEVATIONS IMPERIAL GEODETIC AND ALL DIMENSIONS IN FEET.

REV	DATE	REVISION DESCRIPTION	ENG	APPR

W-E-R ENGINEERING LTD.
CALGARY, ALBERTA

PROJECT: LITTLE BOW RIVER BASIN
STORAGE SITE INVESTIGATION
FEASIBILITY DESIGNS

LITTLE BOW SITE 1

GENERAL LAYOUT

DESIGNED	CHECKED	DATE	JOB No.	DRAWING No.
		16-06-'85		C1
DRAFTED L N	APPROVED	SCALE	SHEET No.	

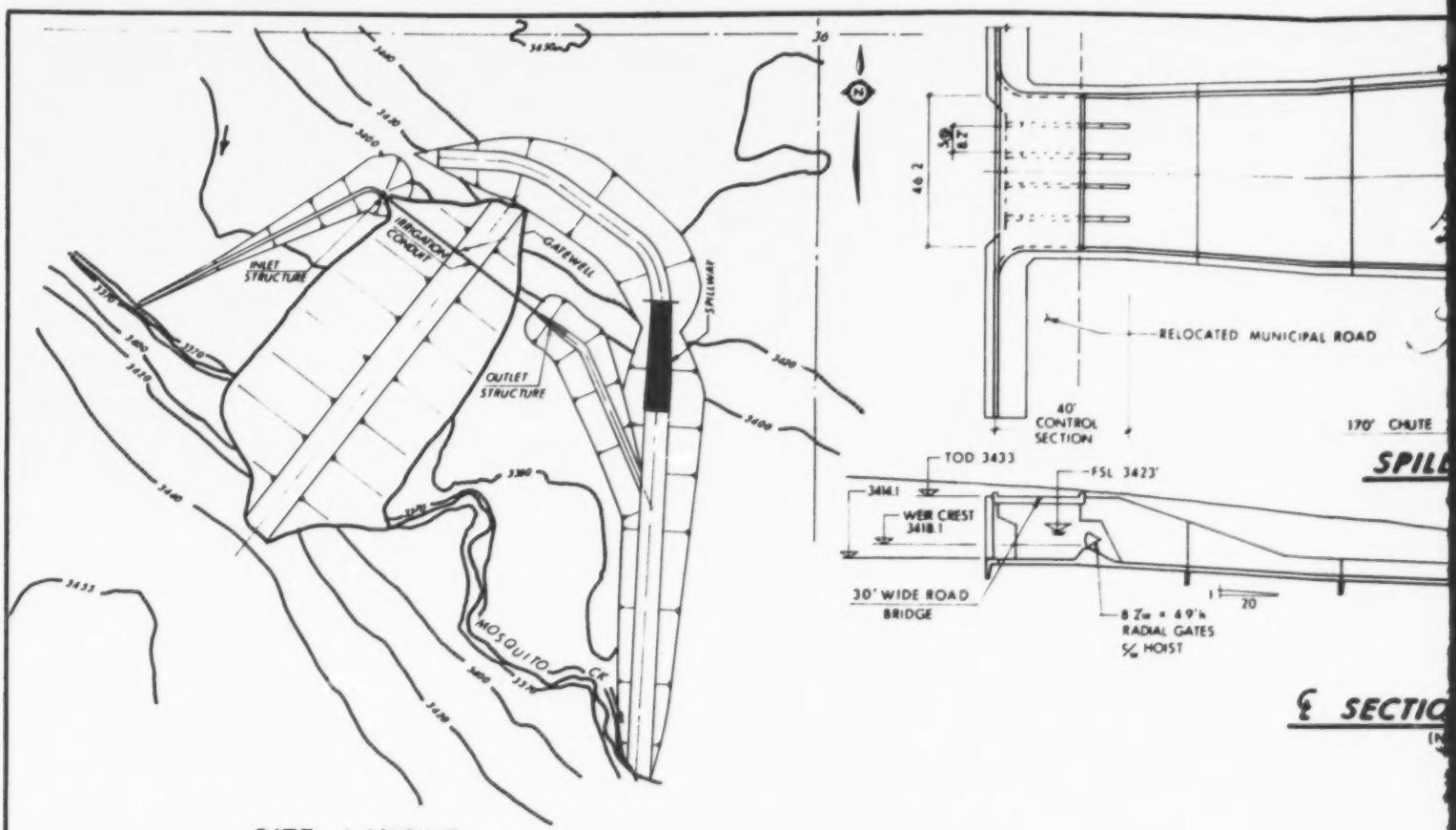
MOSQUITO CREEK SITE 4

TABLE C3
DETAILED COST ESTIMATE
MOSQUITO CREEK SITE 4

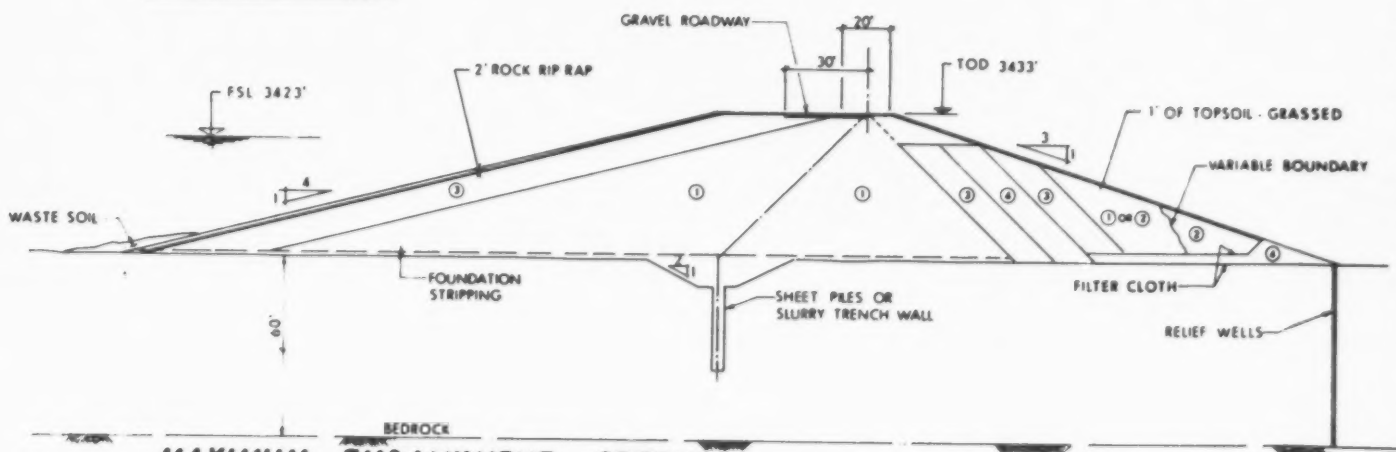
	Quantity	Unit Price	Extension
Care of Water	lump sum		235,000
SUBTOTAL			
Embankment			
Foundation preparation			
- Stripping	15,000 yds^3	1.25	18,750
- Key Trench Excavation	13,500 yds^3	2.00	27,000
- Slurry Trench Excavation	5,900 yds^3	3.00	17,700
- Slurry Trench Installation	3,500 yds^2	150.00	525,000
- Abutment - shotcreting	250 yds^3	475.00	118,750
Compacted Embankment			
- Zone #1	273,000 yds^3	0.70	191,100
- Zone #2	39,000 yds^3	0.70	27,300
Embankment Filter and Drainage System			
- Zone #3	107,000 yds^3	0.70	74,900
- Zone #4	27,000 yds^3	16.00	432,000
- Perf. PVC Pipe	600 ft.	11.00	6,600
- 8" dia. Pressure Relief Wells	240 ft.	50.00	12,000
- Filter Cloth	17,900 yds^2	1.25	22,375
Borrow Area Excavation			
- Zone #1	389,000 yds^3	1.50	583,000
Rock Riprap	14,200 yds^3	27.00	383,400
Topsoil D/S Face of Dam	6,300 yds^3	1.25	7,875
SUBTOTAL EMBANKMENT			2,448,000
Spillway			
Excavation and Haul			
- Rock	16,540 yds^3	5.00	82,700
- Other	135,060 yds^3	2.00	270,120
Zone #1 Backfill	20,000 yds^3	3.00	60,000
Reinforced Concrete	2,290 yds^3	400.00	916,000
Drainage System	731 yds^3	19.00	14,000
Radial Gates & Hoists (5 sets)	lump sum		171,000
Miscellaneous Items			75,700
SUBTOTAL SPILLWAY			1,589,000
Irrigation Supply Conduit and Diversion Works			
Excavation - Structures & Pipe	9,300 yds^3	2.00	18,600
Reinforced Concrete (Structural)	500 yds^3	600.00	300,000
Precast Pipe	446 lin.ft.	380.00	169,480
Zone #1 Backfill (Structural around pipe)	8,600 yds^3	3.00	25,800
Control Gates & Hoists	lump sum		51,700
Inlet and Outlet Channel Excavation	31,900 yds^3	2.00	63,800
Structure Drainage Systems	38 yds^3	19.00	700
Miscellaneous Items			31,500
SUBTOTAL IRRIGATION SUPPLY CONDUIT OUTLET WORKS			662,000
SUBTOTAL CONSTRUCTION			4,934,000
Contingencies (20%)			987,000
TOTAL CONSTRUCTION COST			5,921,000

TABLE C4
Pertinent Data
Mosquito Creek Site 4

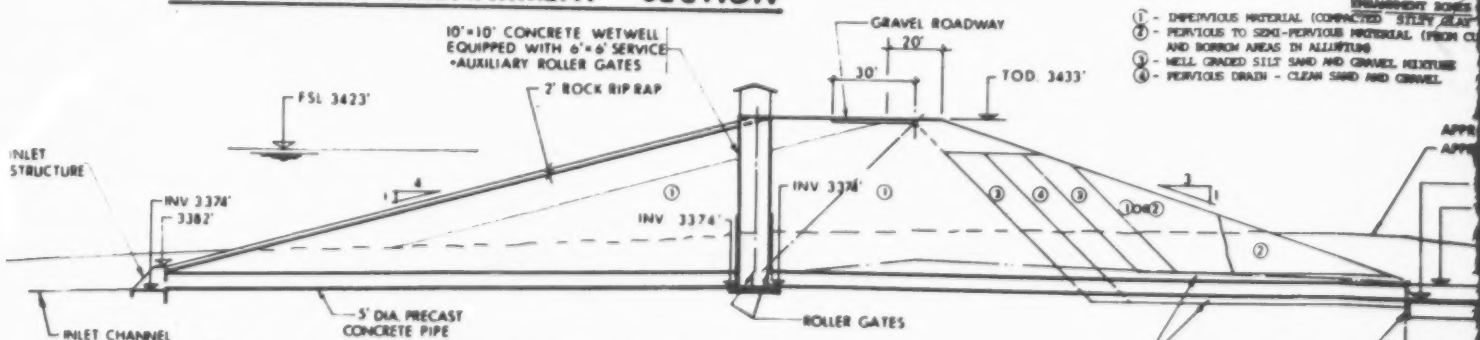
Reservoir	
Full Supply Level (FSL)	3,423 ft.
Capacity at FSL	21,000 ac.ft.
Flooded Area at FSL	1,200 ac.
Maximum Depth at FSL	57 ft.
Probable Maximum Flood (inflow)	15,800 cfs
Project Design Flood - 0.5 PMF (inflow)	7,900 cfs
Maximum Water Level	3,430 ft.
Embankment	
Top of Dam Elevation	3,433 ft.
Crest Length	1,060 ft.
Maximum Height	67 ft.
Volume of Fill	466,000 $yds.^3$
Freeboard at Design Discharge	4.7 ft.
Freeboard at Probable Maximum Discharge	3 ft.
Spillway	
Crest Elevation	3,418.1 ft.
Radial Gates	8.2 ft. W x 4.9 ft. H
Net Weir Length	41 ft.
Gross Crest Width	46.2 ft.
Basin Width	70 ft.
Structure Length	258 ft.
Basin Elevation	3,366 ft.
Design Discharge	5,000 cfs
Probable Maximum Discharge	6,320 cfs
Irrigation Supply Conduit	
Size of Conduit	5 ft. dia.
Length	446 ft.
Irrigation Supply Capacity Requirement	300 cfs
Gate Size	6' x 6'
Intake Elevation	3,374 ft.
Basin Elevation	3,366 ft.
Minimum Reservoir Elevation to Pass Irrigation Requirement	3,388 ft.



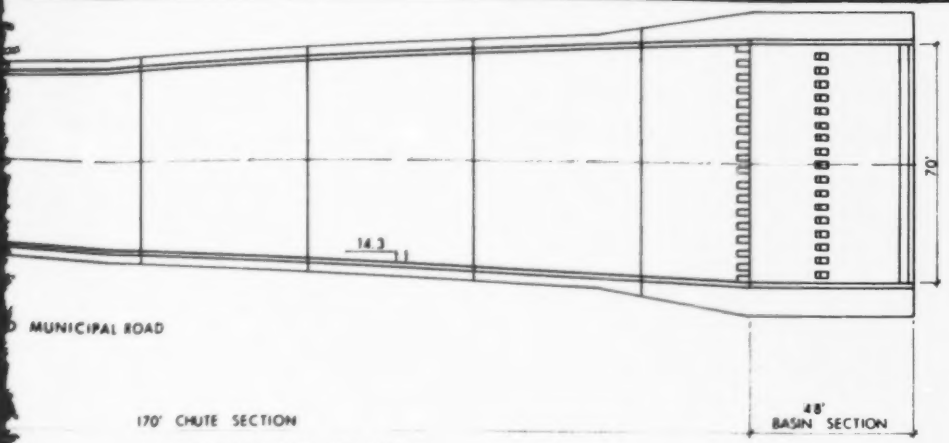
SITE LAYOUT



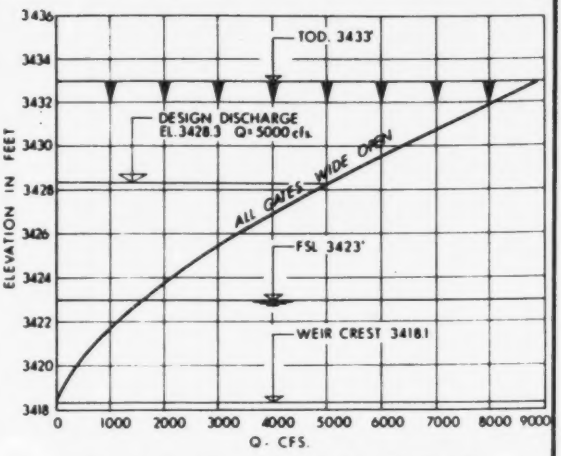
MAXIMUM EMBANKMENT SECTION



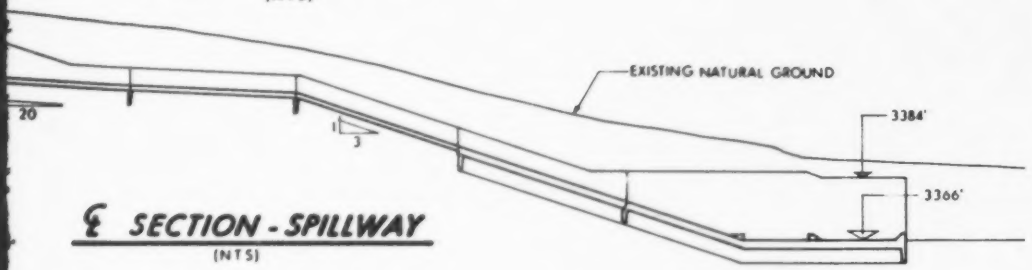
SECTION - IRRIGATION SUPPLY CONDUIT AND DIVERSION WORKS



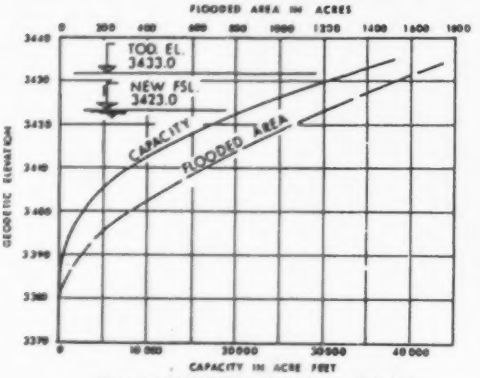
SPILLWAY PLAN
(NTS)



SPILLWAY RATING CURVE



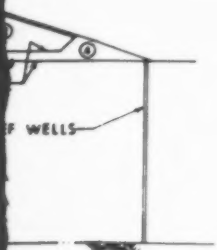
℄ SECTION - SPILLWAY
(NTS)



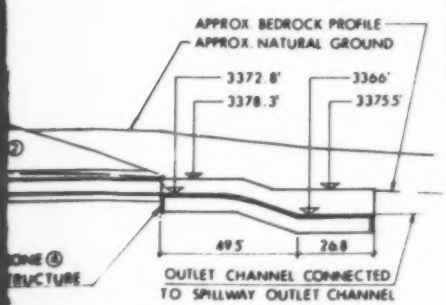
FLOODED AREA AND CAPACITY CURVES

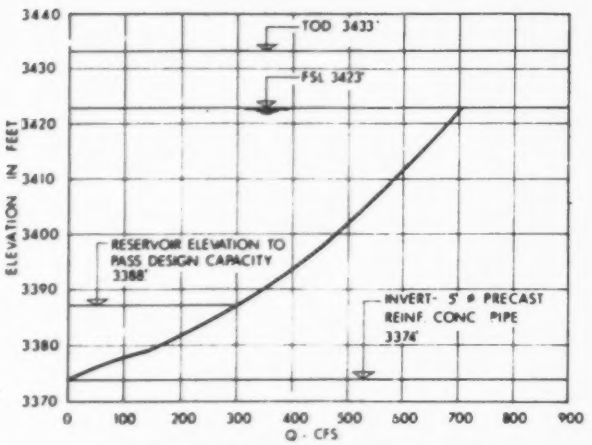
TOPSOIL - GRASSED

VARIABLE BOUNDARY



EMBANKMENT ZONES
TERIAL (COMPACTED SILTY CLAY FROM UPLANDS)
SEMI-PERVIOUS MATERIAL (FROM CUTOFF EXCAVATION
EAS IN ALLUVIUM)
SILT SAND AND GRAVEL MIXTURE
N - CLEAN SAND AND GRAVEL





IRRIGATION SUPPLY CONDUIT RATING CURVE

NOTE. ALL ELEVATIONS IMPERIAL GEODETIC AND ALL DIMENSIONS IN FEET.

REV	DATE	REVISION DESCRIPTION	ENG	APPR

water W-E-R ENGINEERING LTD.
CALGARY, ALBERTA

PROJECT: LITTLE BOW RIVER BASIN
STORAGE SITE INVESTIGATION
FEASIBILITY DESIGNS

MOSQUITO CREEK SITE 4

GENERAL LAYOUT

DESIGNED	CHECKED	DATE	JOB No.	DRAWING No.
		14-06-'85		C 2
DRAFTED L N	APPROVED	SCALE	SHEET No.	

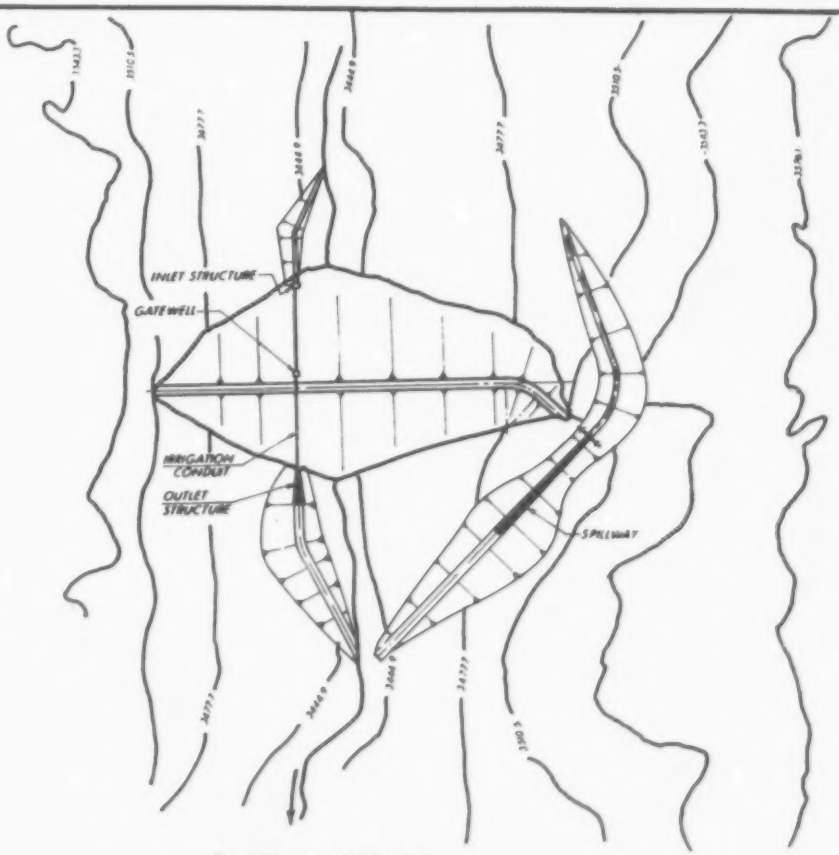
SQUAW COULEE SITE 2

TABLE C5
DETAILED COST ESTIMATE
SQUAW COULEE SITE 2

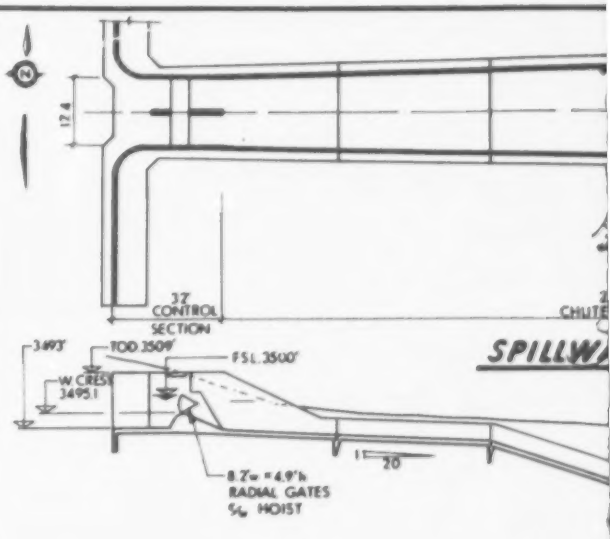
	Quantity	Unit Price	Extension
Care of Water	lump sum		174,000
SUBTOTAL			
Embankment			
Foundation preparation			
- Stripping	12,900 yds^3	1.25	16,125
- Key Trench Excavation	40,200 yds^3	2.00	80,400
- Abutment - shotcreting	220 yds^3	475.00	104,500
Compacted Embankment			
- Zone #1	286,300 yds^3	0.70	200,410
- Zone #2	92,800 yds^3	0.70	64,960
Borrow Excavation	405,500 yds^3	1.50	608,250
Embankment Filter and Drainage System			
- Zone #4	15,500 yds^3	16.00	248,000
- Filter Cloth	21,200 yds^2	1.25	26,500
Rock Riprap	13,300 yds^3	27.00	359,100
Riprap Bedding	11,200 yds^3	12.00	134,400
Topsoil D/S Face of Dam	4,700 yds^3	1.25	5,875
SUBTOTAL EMBANKMENT			1,848,000
Spillway			
Excavation and Haul			
- Rock	30,000 yds^3	5.00	150,000
- Other	117,600 yds^3	2.00	235,200
Zone #1 Backfill	20,000 yds^3	3.00	60,000
Reinforced Concrete	1,180 yds^3	400.00	472,000
Drainage System	300 yds^3	19.00	5,700
Radial Gates & Hoists (2 sets)	lump sum		68,300
Miscellaneous Items			62,700
SUBTOTAL SPILLWAY			1,054,000
Irrigation Supply Conduit and Outlet Works			
Excavation - Structural	1,300 yds^3	2.00	2,600
Inlet and Outlet Channel Excavation	13,200 yds^3	2.00	26,400
Zone #1 Backfill	850 yds^3	3.00	2,550
Structural Concrete	500 yds^3	600.00	300,000
Control Gates & Hoists	lump sum		51,700
Structure Drainage Systems	40 yds^3	19.00	760
Reinforced Concrete Pipe	430 lin. ft.	380.00	163,400
Miscellaneous Items			27,370
SUBTOTAL IRRIGATION SUPPLY CONDUIT OUTLET WORKS			575,000
SUBTOTAL CONSTRUCTION			3,651,000
Contingencies (20%)			730,000
TOTAL CONSTRUCTION COST			4,381,000

TABLE C6
Pertinent Data
Squaw Coulee Site 2

Reservoir	
Full Supply Level (FSL)	3,500 ft.
Capacity at FSL	14,400 ac.ft.
Flooded Area at FSL	600 ac.
Maximum Depth at FSL	64 ft.
Probable Maximum Flood (inflow)	5,900 cfs
Project Design Flood - 0.5 PMF (inflow)	2,950 cfs
Maximum Water Level	3,504.3 ft.
Embankment	
Top of Dam Elevation	3,509 ft.
Crest Length	1,010 ft.
Maximum Height	73 ft.
Volume of Fill	440,000 $yds.^3$
Freeboard at Design Flood	8 ft.
Freeboard at Probable Maximum Flood	4.7 ft.
Spillway	
Crest Elevation	3,495.1 ft.
Radial Gates	8.2 ft. W x 4.9 ft. H
Net Weir Length	16.4 ft.
Gross Crest Width	17.4 ft.
Basin Width	30 ft.
Structure Length	311 ft.
Basin Elevation	3,437 ft.
Design Discharge	900 cfs
Probable Maximum Discharge	1,770 cfs
Irrigation Supply Conduit	
Size of Conduit	5 ft. dia.
Length	440 ft.
Irrigation Supply Capacity Requirement	300 cfs
Gate Size	6 ft. x 6 ft.
Intake Elevation	3,445 ft.
Basin Elevation	3,436 ft.
Minimum Reservoir Elevation to Pass Irrigation Requirement	3,459 ft.



SITE LAYOUT



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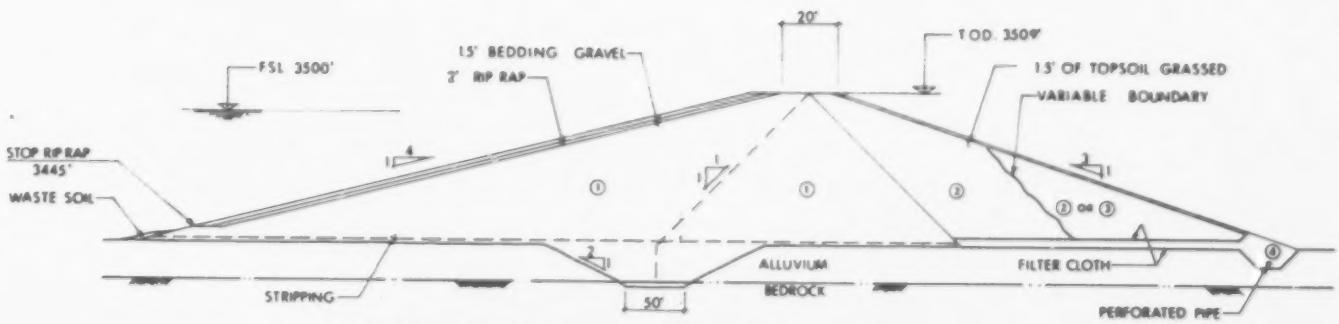
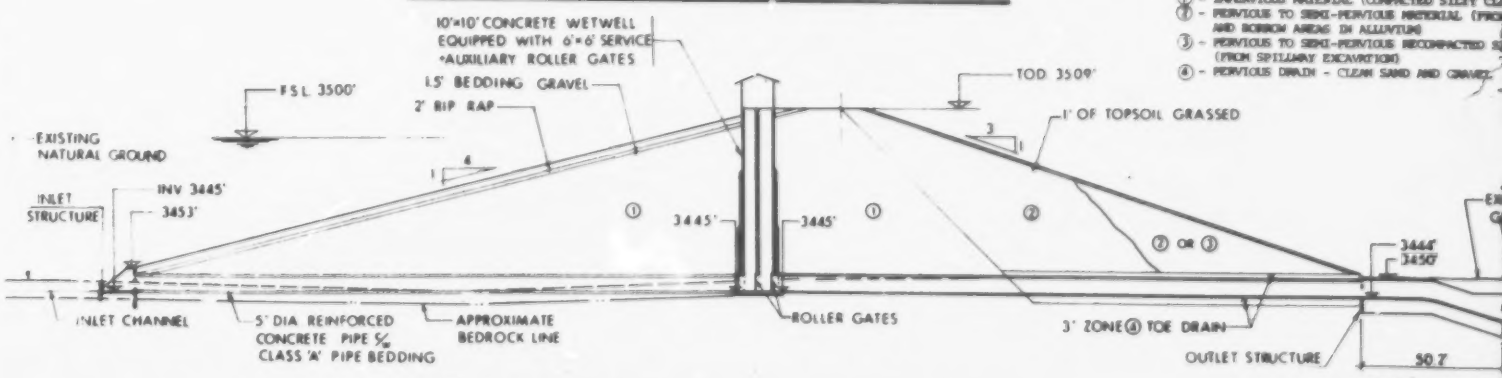


MAXIMUM EMBANKMENT SECTION



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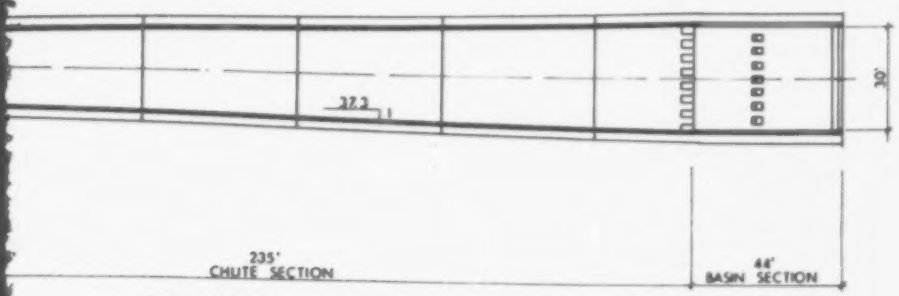
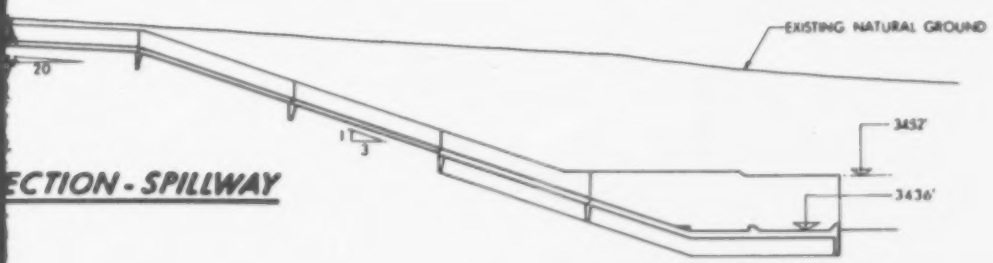
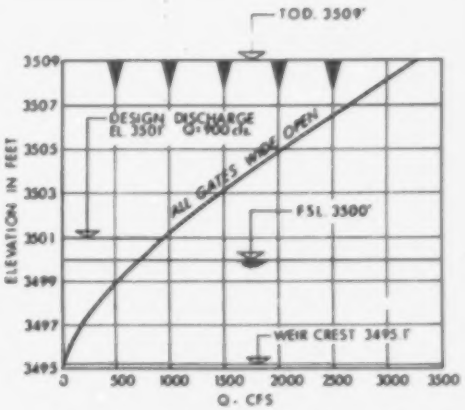


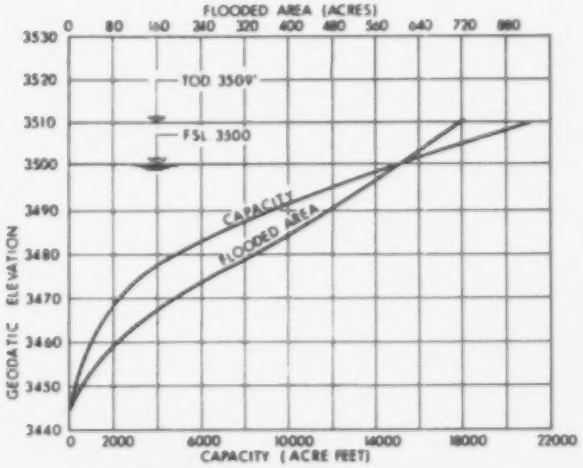
SPILLWAY PLAN



ECTION - SPILLWAY

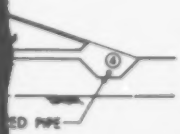


SPILLWAY RATING CURVE

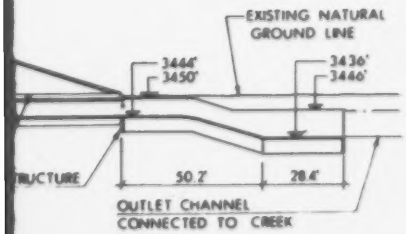


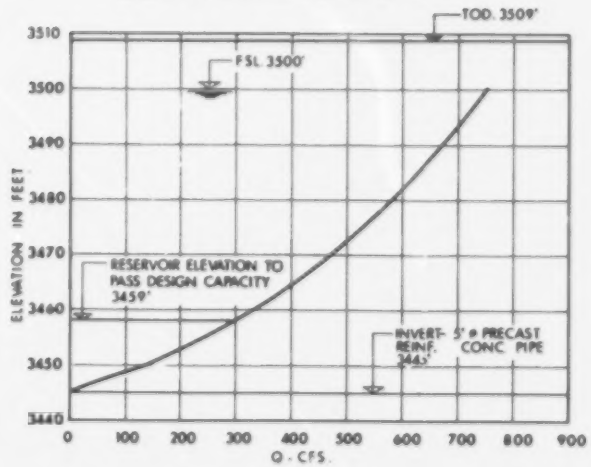
FLOODED AREA AND CAPACITY CURVES

GRASSED
NDARY



EMBANKMENT ZONES
VIOUS MATERIAL (COMPACTED SILTY CLAY FROM UPLANDS)
US TO SEMI-PERVIOUS MATERIAL (FROM CUTOFF EXCAVATION
ROM AREAS IN ALLUVIUM)
US TO SEMI-PERVIOUS RECOMPACTED SANDSTONE+SHALE
SPILLWAY EXCAVATION)
US DRAIN - CLEAN SAND AND GRAVEL





IRRIGATION SUPPLY CONDUIT RATING CURVE

NOTE: ALL ELEVATIONS IMPERIAL GEODETIC AND ALL DIMENSIONS IN FEET.

REV	DATE	REVISION DESCRIPTION	ENG	APPR

W-E-R ENGINEERING LTD.
CALGARY, ALBERTA

PROJECT: LITTLE BOW RIVER BASIN
STORAGE SITE INVESTIGATION
FEASIBILITY DESIGNS

SQUAW COULEE SITE 2

GENERAL LAYOUT

DESIGNED	CHECKED	DATE	JOB No.	DRAWING No.
		15-06-'85		C 3
DRAFTED LN	APPROVED	SCALE	SHEET No.	

LITTLE BOW RIVER BASIN
STORAGE SITE INVESTIGATION
FEASIBILITY DESIGN REPORT
June, 1985

APPENDIX D

RESERVOIR COSTS

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1.0	BASIS OF RESERVOIR COSTING	D-1
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	1.2 Reservoir Damages	D-1
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	2.1 Land Acquisition	D-3
	2.2 Reservoir Damages	D-3
	2.2.1 Roads	D-3
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3.0	MOSQUITO CREEK SITE 4	
	3.1 Land Acquisition	D-5
	3.2 Reservoir Damages	D-5
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4.0	SQUAW COULEE SITE 2	
	4.1 Land Acquisition	D-9
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	4.2.1 Roads and Bridges	D-9
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Title	Table No.
Reservoir Costs - Little Bow Site 1	D1
Reservoir Costs - Mosquito Creek 4	D2
Reservoir Costs - Squaw Coulee 2	D3

LIST OF DRAWINGS

Title	Drawing No.
Little Bow Site 1 - Reservoir Damages Summary Plan	D1
Mosquito Creek Site 4 - Reservoir Damages Summary Plan	D2
Squaw Coulee Site 2 - Reservoir Damages Summary Plan	D3

1.0 BASIS OF RESERVOIR COSTING

An estimate of the costs of land acquisition and reservoir damages has been prepared for each of the three storage sites, as detailed in subsequent sections of this report.

1.1 Land Acquisition

The areas of lands required for reservoir land control were based on top-of-dam elevation to allow for possible future erosion on the reservoir abutments. The breakdown of land use was based on field inspections and an air photo appraisal (1983 photos) in conjunction with reservoir topographic plans. Unit land costs of $800 per acre for cultivated lands and $300 for grassland, used in the conceptual design report were discussed with officials from the Municipal District of Foothills. Based on the discussions they are considered to be reasonable and are used in this report. An additional 25% for severance and acquisition costs has been added.

1.2 Reservoir Damages

Damages associated with construction of the storage projects included in this phase of the study involve the following:

- raising and enlargement or relocation of existing roads and bridges;
- power transmission lines;
- telephone lines;
- natural gas lines;
- relocation or dyke protection of farm buildings.

The costs were estimated based on information supplied by TransAlta Utilities Corporation, Canadian Western Natural

Gas Company Limited, and Alberta Government Telephones. Details of the reservoir costs for each site are included in subsequent sections of this report.

The costs associated with relocation of farm buildings and connection of utilities were based on costs supplied by moving companies in connection with a study of Keho Lake Enlargement (1980) updated to 1985.

2.0 LITTLE BOW SITE 1

2.1 Land Acquisition

The total area of lands flooded to the top-of-dam elevation is 892 acres, of which 250 acres are cultivated or irrigated, and 642 acres are in grass.

The cost of acquisition is estimated as follows:

Cultivation or Irrigation	250 ac. @ $800 =	$200,000
Other	642 ac. @ $300 =	193,000
Severance (25%)		98,000
	TOTAL	$491,000

2.2 Reservoir Damages

2.2.1 Roads

(a) East Boundary Sec. 3-18-27-W4

An existing gravel road crosses the Little Bow River via a pile supported concrete bridge 72 feet long by 24 feet wide. The road would have to be raised by five feet and the slope armoured with riprap. The existing bridge deck would have to be raised and a new sub-structure constructed.

The costs of this road crossing are estimated as follows:

Embankment	7400 yd^3 @ $2.00	14,800
Bedding Gravel	1215 yd^3 @ $12.00	14,600
Riprap	1800 yd^3 @ $27.00	48,600
Road Gravel	400 yd^3 @ $12.00	5,000
Bridge Raising	lump sum	150,000
		233,000
	Contingencies (25%)	58,000
	TOTAL	$291,000

2.2.2 Farm Buildings

Three sets of farm buildings, in the NE 36-17-28-W4, NW 1-18-28-W4, and SE 2-18-28-W4, are located within the reservoir area. The costs of relocation including re-connection of utilities are estimated at $75,000 each for a total cost of $225,000.

2.2.3 Utilities

No power lines, pipelines or telephone lines have been identified that would be affected by the reservoir except for service to those farmsteads which would be relocated. The costs of re-connection are included with the lump sum relocation costs.

TABLE D1
RESERVOIR DAMAGES
LITTLE BOW SITE 1

Land Acquisition	884,000
Roads	291,000
Farmstead Relocations	225,000
TOTAL	$1,400,000

3.0 MOSQUITO CREEK SITE 4

3.1 Land Acquisition

The total area of lands affected by the reservoir is 1,600 acres, of which approximately 580 acres are either cultivated or irrigated and the remainder is in grassland.

The cost of acquisition is estimated as follows:

Cultivation or Irrigation	580 ac. @ $800	464,000
Other	950 ac. @ $300	285,000
Severance (25%)		187,000
TOTAL		$936,000

3.2 Reservoir Damages

3.2.1 Roads and Bridges

(a) North Boundary Sec. 35-16-29-W4

The existing road and bridge, if retained at their present location would have to be raised by approximately 38 feet. Preliminary calculations indicate that tne cost of this alternative would be prohibitive. Alternatively, the road can be feasibly re-routed over the top of the dam, at a cost estimated as follows:

Embankment	9,900 yd^3 @ $2.00	20,000
Road Gravel	7,000 yd^3 @ $12.00	84,000
	Contingencies (25%)	104,000
		26,000
		$130,000

(Note: spillway costs included under main project costs)

Land Acquisition		
	36 acres @ $800	29,000
	Severance (25%)	7,000
		$ 36,000
	TOTAL COST	$166,000

(b) North Boundary Sec. 11-17-29-W4

This road and bridge would have to be raised by some 15 feet at a cost, including slope armouring, estimated as follows:

Embankment	51,000 yd^3 @ $2.00	102,000
Bedding Gravel	4,970 yd^3 @ $12.00	59,600
Riprap	7,455 yd^3 @ $27.00	201,300
Road Gravel	800 yd^3 @ $12.00	9,600
Bridge Costs	lump sum	325,000
	Contingencies (25%)	174,000
	TOTAL	872,000

(c) West Boundary Sec. 1-17-29-W4

This road would have to be raised for approximately one mile and a drainage pumpstation constructed in the low area on the east side of the road to pick up local runoff.

Embankment		172,000
Slope Armouring		125,000
Road Gravel		10,000
Drainage Pumpstation		90,000
	Contingencies (25%)	99,000
	TOTAL	496,000

3.2.2 Farmsteads

Three farmsteads would be flooded by the reservoirs.

(a) Buildings in NW 36-16-29-W4 could be feasibly dyked off from the reservoir and retained in place.

The costs are estimated as follows:

Embankment	11,900 yd^3 @ $2.00	23,800
Riprap	960 yd^3 @ $27.00	26,000
Bedding Gravel	960 yd^3 @ $12.00	11,500
	Contingencies (25%)	15,300
	TOTAL	$77,000

(b) Buildings in SE 14-17-29-W4 could be feasibly dyked off and left in place, at a cost estimated as follows:

Embankment	10,400 yd^3 @ $2.00	20,800
Riprap	940 yd^3 @ $27.00	25,400
Bedding Gravel	940 yd^3 @ $12.00	11,300
	Contingencies (25%)	14,300
	TOTAL	$72,000

(c) Buildings in NE 2-17-29-W4 would have to be relocated at an estimated cost of $75,000.

3.2.3 Power Lines

(a) 500 KVA Power Line North Boundary

Section 2-17-29-4

Trans-Alta estimates the cost of relocating this line to

south of reservoir at $3,000,000. Alternatively, special long-span support tower might be used if minimum ground or water surface clearance of 35 feet can be maintained. The estimated minimum cost of this alternative according to Trans-Alta is $1,000,000.

(b) Small Power Line, North Boundary Sec. 11-17-29-W4

A small power line supported on wooden poles crosses the reservoir at the above location. Relocation around the north end of the reservoir would be the cheapest alternative at an estimated cost of

2 miles @ $10,000/mile = $20,000.

3.2.4 Pipelines

(a) A 50 mm gas pipeline crosses reservoir in Sections 35 and 36-16-29-4.

Relocation costs are estimated as follows:

1.1 miles @ $25,000/mile = $27,500

(b) A small farm service line in Sections 9 and 10-17-29-W4 would require about 1.0 miles of relocation.

Cost of $15,000 = $15,000

3.2.5 Telephones

(a) A buried telephone cable crosses the reservoir at the North boundary Section 11-17-29-4.

Relocation of 3 miles at $15,000/mile = $45,000

TABLE D2
RESERVOIR DAMAGES
MOSQUITO CREEK SITE 4

Land Acquisition		936,000
Roads	(a)	166,000
	(b)	872,000
	(c)	496,000
Farmstead Protection		149,000
Farmstead Relocation		75,000
Powerline Relocations	(a)	1,000,000
(minimum costs)	(b)	20,000
Pipeline Relocations		43,000
Telephone Line Relocation		45,000
TOTAL		$3,802,000

4.0 SQUAW COULEE SITE 2

4.1 Land Acquisition

The total area of land included in the Squaw Coulee Site 2 reservoir to top-of-dam elevation is 730 acres, of which all but 30 acres are in grassland.

The cost of acquisition is estimated as follows:

Cultivation or Irrigation	30 ac. @ $800	24,000
Other	700 ac. @ $300	210,000
Severance (25%)		59,000
TOTAL		$293,000

4.2 Reservoir Damages

4.2.1 Roads and Bridges

(a) S.H. 540, North Boundary Sec. 34-17-29-W4

The existing road would have to be raised by approximately 33 feet above its present elevation at the maximum section, and a new sectional plate pipe arch installed. The costs are estimated as follows:

Embankment	185,000 yd^3 @ $2.00	370,000
Riprap	15,700 yd^3 @ $27.00	423,900
Bedding Gravel	10,500 yd^3 @ $12.00	126,000
Blacktop Surfacing	1 lump sum	60,000
Sectional Plate Arch Culvert	1 lump sum	91,000
Contingencies (25%)		268,000
TOTAL		1,339,000

(b) Road Crossing North Boundary Sec. 10-18-29-W4

The existing road would require raising by about 23 feet at the maximum section, and a new sectional-plate pipe arch installed.

The cost is estimated as follows:

Embankment	72,100 yd^3 @ $2.00	144,000
Riprap	9,516 yd^3 @ $27.00	257,000
Bedding Gravel	6,344 yd^3 @ $12.00	76,000
Road Gravel	613 yd^3 @ $12.00	7,400
Sectional Plate Arch Culvert		75,000
	Contingencies (25%)	140,000
	TOTAL	699,000

4.2.2 Farm Buildings

No farm buildings would be affected by the Squaw Coulee Site 2 Reservoir.

4.2.3 Pipelines

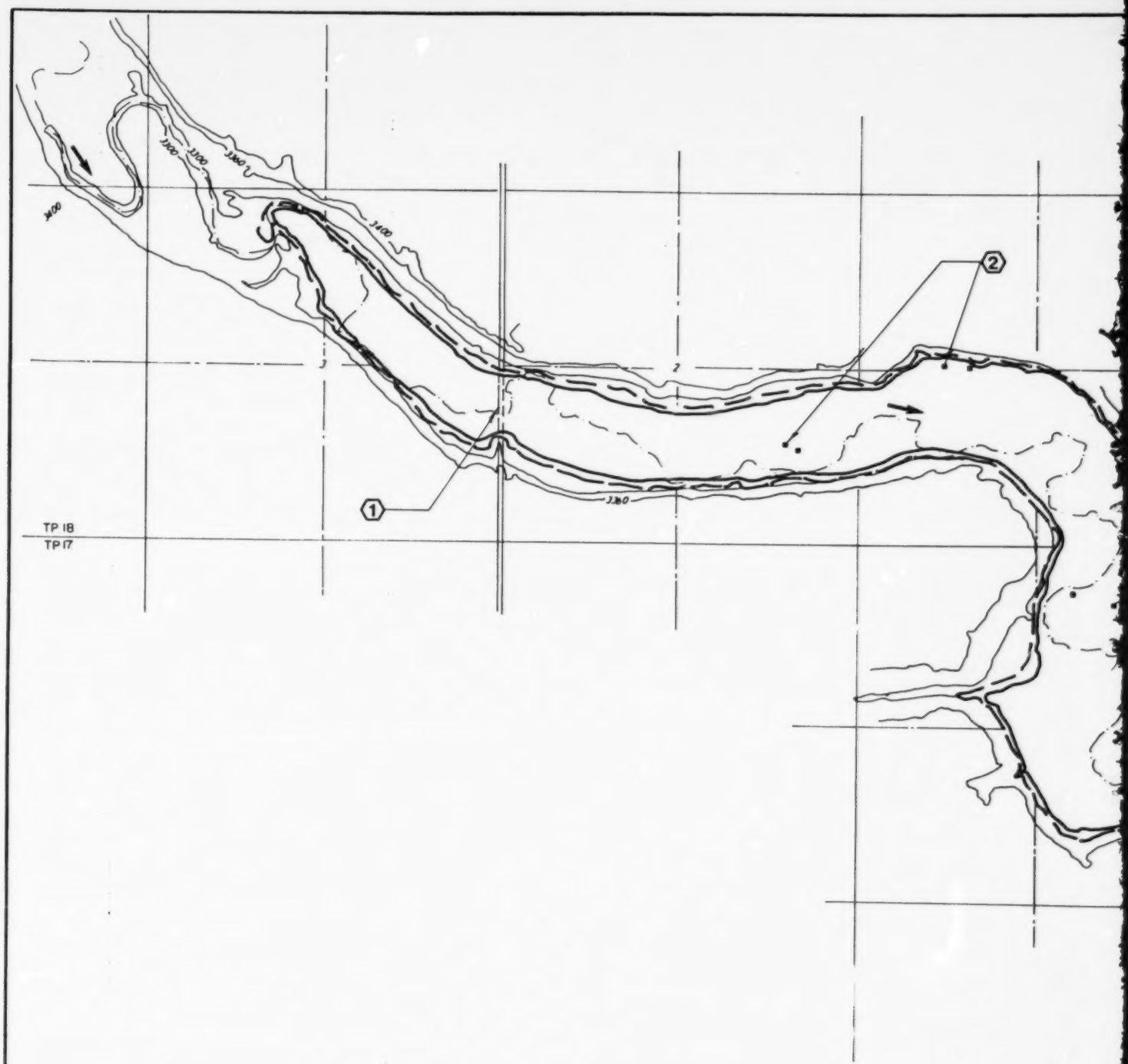
The reservoir would affect gas distribution pipelines operated by Canadian Western Natural Gas (CWNG) in:

E 1/2 34-17-29-W4
S 1/2 15-18-29-W4
S 1/2 3-18-29-W4

The total cost to relocate and/or upgrade the aforementioned pipelines would be $75,000 as quoted by Canadian Western Natural Gas.

TABLE D3
RESERVOIR DAMAGES
SQUAW COULEE SITE 2

Land Acquisition	293,000
Roads (a)	1,339,000
(b)	699,000
Pipeline Relocations and/or Upgrading	75,000
TOTAL	$2,406,000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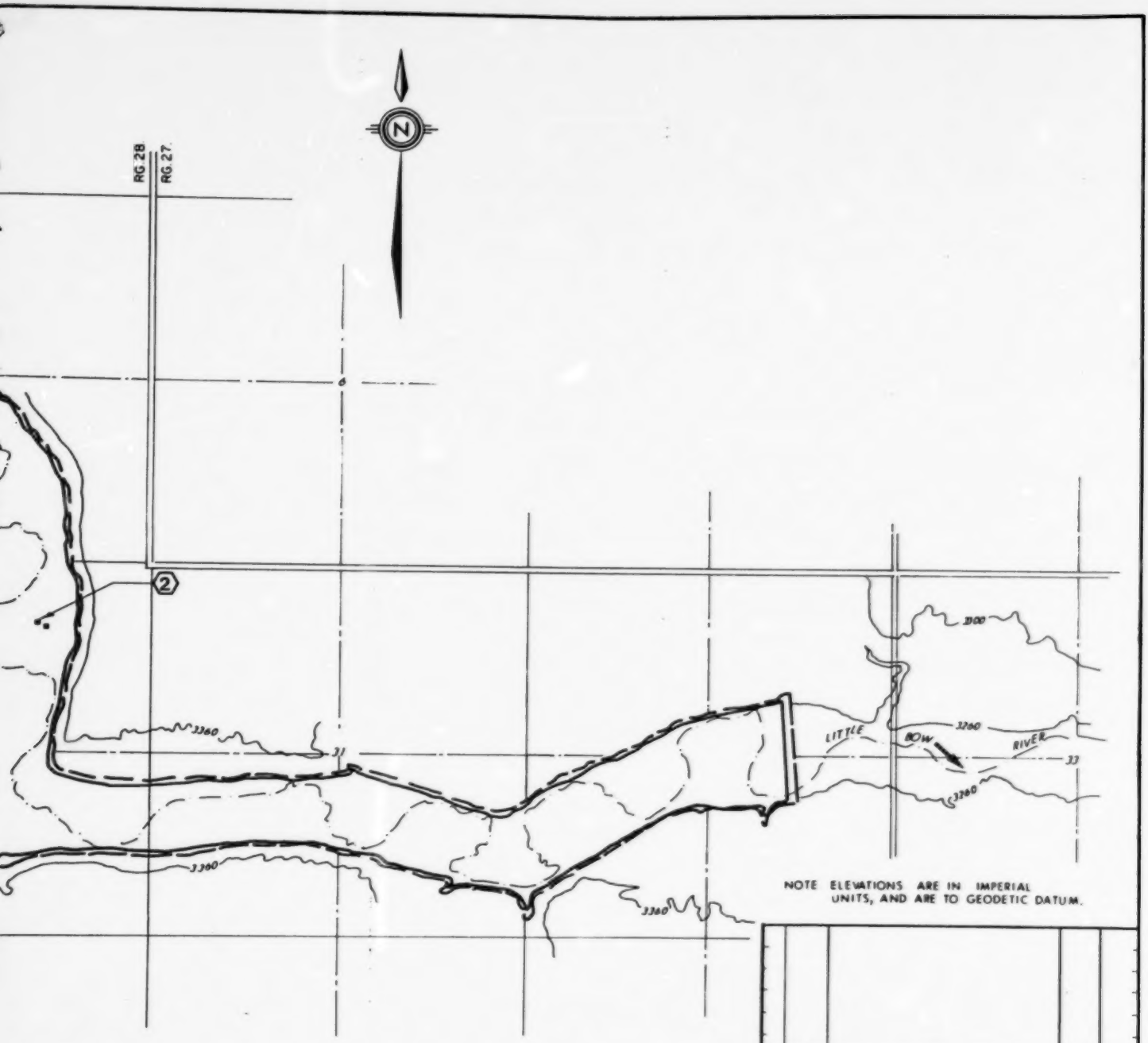
TP 17



NOTE ELEVATIONS ARE IN IMPERIAL UNITS, AND ARE TO GEODETIC DATUM.

SUMMARY OF DAMAGES

—— FSL. AT 3290 FEET

—— TOD. AT 3299 FEET

(1) RAISE ROAD AND INSTALL ARCH CULVERT

(2) THREE SETS OF FARM BUILDINGS TO BE RELOCATED

0 500 1000 2000 FEET

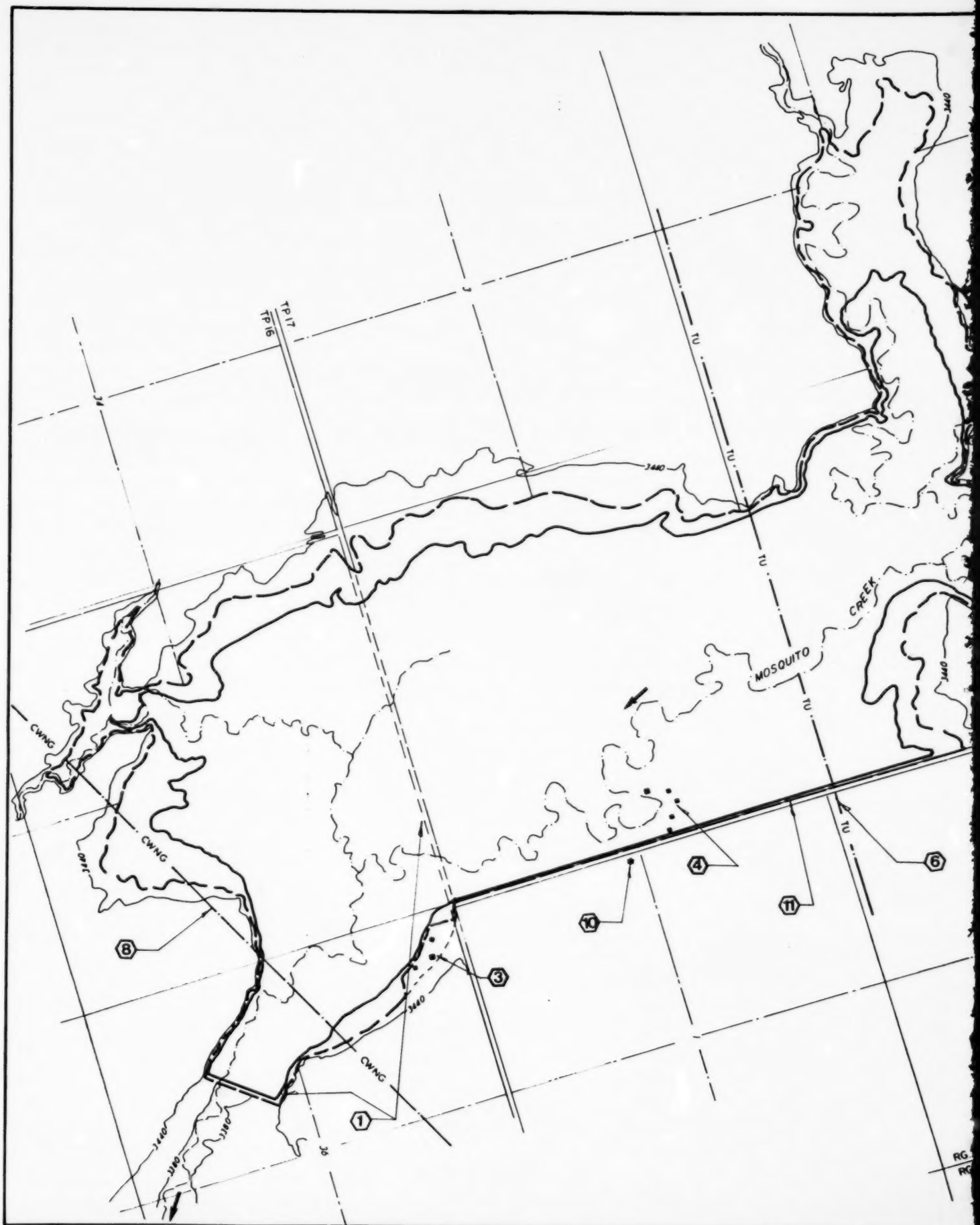
REV	DATE	REVISION DESCRIPTION	ENG	APPR

W-E-R ENGINEERING LTD.
CALGARY, ALBERTA

PROJECT
LITTLE BOW RIVER BASIN
STORAGE SITE INVESTIGATION
FEASIBILITY DESIGNS

LITTLE BOW SITE 1 RESERVOIR DAMAGES SUMMARY PLAN

DESIGNED	CHECKED	DATE	JOB No.	DRAWING No.
B.W.B.	CAR	JUNE 3/85		D1
DRAFTED	APPROVED	SCALE	SHEET No.	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TP16
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TU
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TU
TU
MOSQUITO
CREEK
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3440
3440
3440
3440
3440
CWNG
CWNG
CWNG
DITCH
1
3
4
6
8
10
11
36
RG
RG

SUMMARY OF DAMAGES

—— FSL. AT 3423 FEET

—— TOD. AT 3433 FEET

1. RELOCATE EAST-WEST ROAD TO TOP OF DAM (BRIDGE COST INCLUDED IN SPILLWAY COST)
2. RAISE ROAD AND INSTALL ARCH CULVERT
3. DYKE PROTECTION FOR TWO FARMSTEADS
4. RELOCATE ONE FARMSTEAD
5. RELOCATE ONE MINOR TRANSALTA UTILITIES POWERLINE
6. RAISE OR RELOCATE TRANSALTA 300 KVA POWERLINE
7. CANADIAN WESTERN NATURAL GAS FARM SUPPLY PIPELINE RELOCATE
8. CANADIAN WESTERN NATURAL GAS 50mm ϕ PIPELINE RELOCATE
9. AGT BURIED TELEPHONE CABLE TO BE RELOCATED
10. DRAINAGE PUMPSTATION TO BE CONSTRUCTED
11. RAISE NORTH-SOUTH ROAD AND RIPRAP RESERVOIR FACE

NOTE ELEVATIONS IN IMPERIAL UNITS, AND ARE TO GEODETIC DATUM.

0 500 1000 2000 FEET

REV	DATE	REVISION DESCRIPTION	ENG	APPR

W-E-R ENGINEERING LTD.
CALGARY, ALBERTA

PROJECT
LITTLE BOW RIVER BASIN
STORAGE SITE INVESTIGATION
FEASIBILITY DESIGNS

MOSQUITO CREEK SITE 4 RESERVOIR DAMAGES SUMMARY PLAN

DESIGNED	CHECKED	DATE	JOB No	DRAWING No
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DRAFTED	APPROVED	SCALE	SHEET No	
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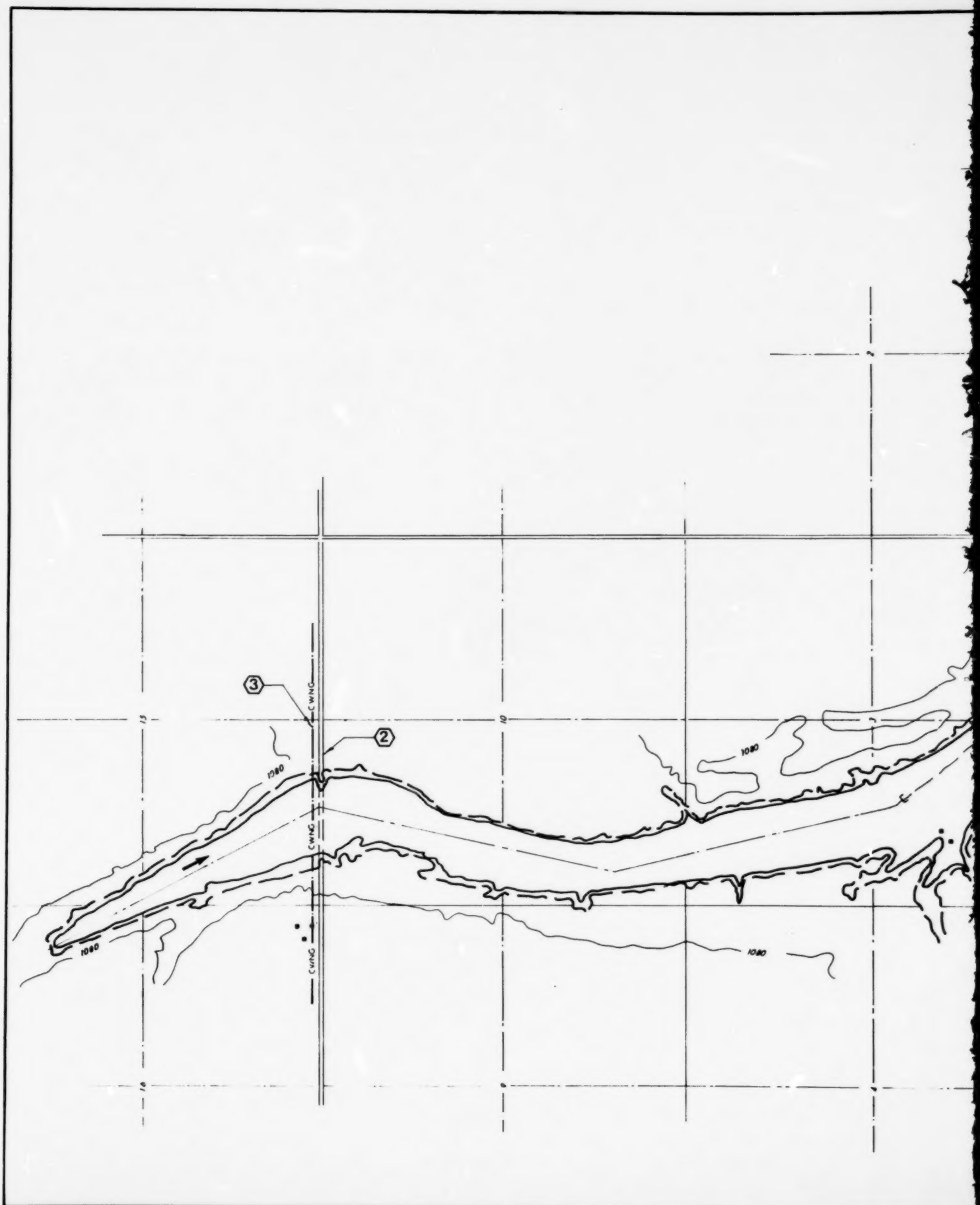
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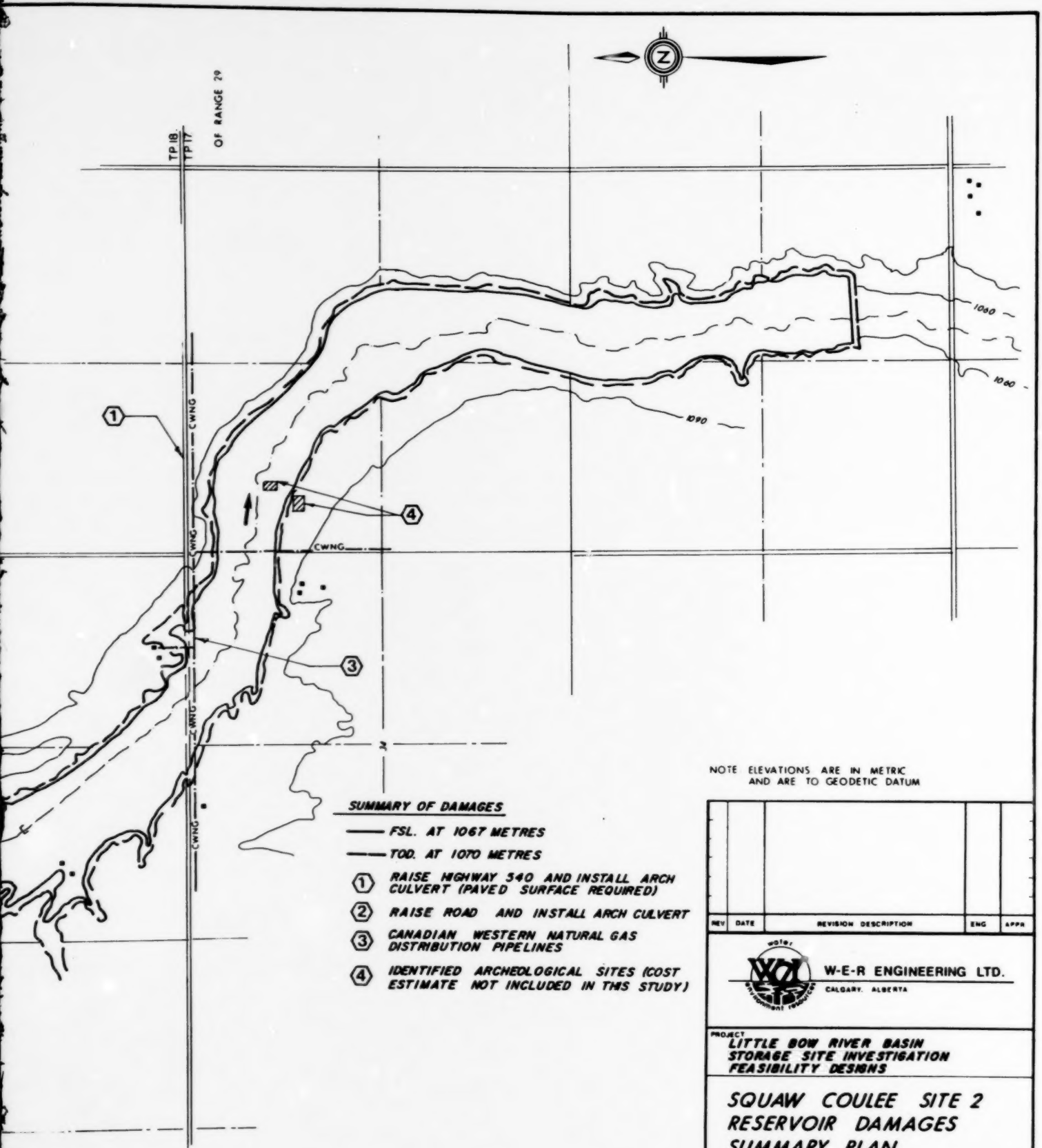

NOTE ELEVATIONS ARE IN METRIC AND ARE TO GEODETIC DATUM

SUMMARY OF DAMAGES

—— FSL. AT 1067 METRES

—— TOD. AT 1070 METRES

① RAISE HIGHWAY 340 AND INSTALL ARCH CULVERT (PAVED SURFACE REQUIRED)

② RAISE ROAD AND INSTALL ARCH CULVERT

③ CANADIAN WESTERN NATURAL GAS DISTRIBUTION PIPELINES

④ IDENTIFIED ARCHEOLOGICAL SITES (COST ESTIMATE NOT INCLUDED IN THIS STUDY)

50
0 100 200 300 400 METRES

REV	DATE	REVISION DESCRIPTION	ENG	APPR

W-E-R ENGINEERING LTD.
CALGARY, ALBERTA

PROJECT
LITTLE BOW RIVER BASIN
STORAGE SITE INVESTIGATION
FEASIBILITY DESIGNS

SQUAW COULEE SITE 2 RESERVOIR DAMAGES SUMMARY PLAN

DESIGNED	CHECKED	DATE	JOB No	DRAWING No
DRAFTED FC	APPROVED	SCALE	SHEET No	D3